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PHASE I INSPECTION REPORT. NATIONAL DAM SAFETY PROGRAM. PACKANA--ETC(U)
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LEVEL II

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PASSAIC RIVER BASIN

BRANCH OF POMPTON RIVER, PASSAIC COUNTY

NEW JERSEY

PACKANACK LAKE DAM

PHASE I INSPECTION REPORT

NATIONAL DAM SAFETY PROGRAM

NJ 00239

DEPARTMENT OF THE ARMY
PHILADELPHIA DISTRICT, CORPS OF ENGINEERS
CUSTOM HOUSE - 2D & CHESTNUT STREETS
PHILADELPHIA, PENNSYLVANIA 19106

FEBRUARY 1978

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18. SUPPLEMENTARY NOTES

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19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

National Dam Safety Program
Dam Inspection Report Phase I
Packanack Lake Dam, N.J.
Dams - N.J.

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report cites results of a technical investigation as to the dam's adequacy. The inspection and evaluation of the dam is as prescribed by the National Dam Inspection Act, Public Law 92-367. The technical investigation includes visual inspection, review of available design and construction records, and preliminary structural and hydraulic and hydrologic calculations, as applicable. An assessment of the dam's general condition is included in the report.

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DEPARTMENT OF THE ARMY
PHILADELPHIA DISTRICT, CORPS OF ENGINEERS
CUSTOM HOUSE—2 D & CHESTNUT STREETS
PHILADELPHIA, PENNSYLVANIA 19106

IN REPLY REFER TO
NAPEN-D

16 JUN 1978

Honorable Brendan T. Byrne
Governor of New Jersey
Trenton, NJ 08621

Dear Governor Byrne:

Inclosed is the Phase I Inspection Report for Packanack Lake Dam in Passaic County, New Jersey which has been prepared under authorization of the Dam Inspection Act, Public Law 92-367. A brief assessment of the dam's condition is given on pages 1 and 2 of the report. This assessment indicates the dam's earth embankment to be in generally fair condition, however, the dam's spillway is considered inadequate.

The inspection notes deficiencies requiring prompt action to insure the adequacy of the facility. The following items, as a minimum, require investigation and/or corrective action by the owner:

a. Determination of the condition of the sewer line within the embankment should be undertaken within three months after the date of approval of this report and a study initiated within six months of the date of the approval of this report to determine the need and feasibility of removing all utilities from the embankment.

b. A hydraulic and hydrologic investigation should be initiated within six months of the date of approval of the report to determine if any corrective construction is required to increase the capacity of the spillway and/or to raise the height of the dam. Concurrent studies should be made to determine the need for replacement of the existing bridge over the spillway with newer and fewer piers.

c. Within one year from the date of the approval of this report, the existing valve at the downstream end of the 20 inch drain pipe through the embankment should be replaced by a valve at the upstream end of the pipe.

d. Work should be initiated within one year of the date of approval of this report to remove tree and brush growth from both slopes of the dam and to repair all areas of local erosion.

NAPEN-D
Honorable Brendan T. Byrne

Two copies of the report are being furnished to Mr. Dirk C. Hofman, New Jersey Department of Environmental Protection, the designated State Office contact for this program. Within five days of the date of this letter, a copy will also be sent to Congressman Robert A. Roe of the Eighth District. Under the provisions of the Freedom of Information Act, the inspection report will be subject to release by this office, upon request, thirty days after the date of this letter.

An important aspect of the Dam Safety Program will be the implementation of the recommendations made as a result of the inspection. We accordingly request that we be advised of proposed actions taken by the State to implement our recommendations.

Sincerely yours,

Harry V Dutchyshyn

HARRY V. DUTCHYSHYN
Colonel, Corps of Engineers
District Engineer

1 Incl
As stated

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Mr. Dirk C. Hofman, P.E.
Department of Environmental Protection

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PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam Packanack Lake Dam

State Located New Jersey
County Located Passaic County
Stream Packanack Brook
Date of Inspection January 12, 1978

ASSESSMENT OF GENERAL CONDITIONS

The general condition of the earth embankment of Packanack Dam is fair. No significant seepage through or settlement of the embankment was noted during the visual inspection. However, erosion of the embankment in the vicinity of the spillway is occurring which, if allowed to continue, may threaten the embankment or cause a blockage of the spillway. The embankment is heavily covered with heavy brush and trees up to twelve inches in diameter. Some localized areas of erosion and minor settlement were noted in the embankment. Utility lines are located in the downstream part of the embankment and in the road immediately downstream of the dam.

The side slopes of the embankment are relatively steep but the difference in elevation between the normal reservoir surface and the ground downstream of the dam is only eight feet. The freeboard between the normal reservoir surface and the top of the dam is six feet.

The embankment is constructed of impervious earth materials on the upstream side, a puddled clay central core and more pervious materials on the downstream side. In view of the materials used in construction, the relatively low height of the dam and the lack of visible seepage or slope instability on the downstream side of the dam, the side slopes appear to be adequate.

Three piers which support a bridge are located on the spillway. In addition, concrete stoplog supports (one foot wide by one foot high) are located in the center of each spillway span. The bridge piers and stoplog supports may collect debris during a flood and reduce the spillway capacity.

Cracks in the concrete bridge piers as well as erosion of the piers were noted during the inspection. Failure of the bridge could cause a blockage of the

spillway in addition to the other hazards normally associated with bridge failures.

Erosion is occurring in the outlet channel immediately downstream of the spillway. An eighteen inch diameter sanitary sewer line crosses the outlet channel immediately downstream of the spillway and appears to be an obstruction to spillway flow. Wing walls in the outlet channel reduce the width of the channel from sixty feet at the spillway to fifteen feet downstream of the spillway.

The spillway is unable to pass a storm equivalent to one half the Probable Maximum Flood (PMF) without overtopping the dam. One half the PMF is the spillway design flood for this dam according to the Recommended Guidelines for the Inspection of Dams.

John J. Williams

John J. Williams, P.E.
Vice President

The inspection indicates deficiencies requiring prompt action to insure the adequacy of the facility. The following items, as a minimum, require investigation and/or corrective action by the owner:

a. Determination of the condition of the sewer line within the embankment should be undertaken within three months after the date of approval of this report and a study initiated within six months of the date of the approval of this report to determine the need and feasibility of removing all utilities from the embankment.

b. A hydraulic and hydrologic investigation should be initiated within six months of the date of approval of the report to determine if any corrective construction is required to increase the capacity of the spillway and/or to raise the height of the dam. Concurrent studies should be made to determine the need for replacement of the existing bridge over the spillway with newer and fewer piers.

c. Within one year from the date of the approval of this report, the existing valve at the downstream end of the 20 inch drain pipe through the embankment should be replaced by a valve at the upstream end of the pipe.

d. Work should be initiated within one year of the date of approval of this report to remove tree and brush growth from both slopes of the dam and to repair all areas of local erosion.

APPROVED: Harry V Dutchyshyn
HARRY V. DUTCHYSHYN
Colonel, Corps of Engineers
District Engineer

DATE: 8 June 1978

OVERALL VIEW OF DAM, SPILLWAY AND BRIDGE OVER SPILLWAY

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
NAME OF DAM PACKANACK LAKE DAM ID# NJ00239

SECTION 1 - PROJECT INFORMATION

1.1 GENERAL

a. Authority

This report is authorized by the Dam Inspection Act, Public Law 92-367, and has been prepared in accordance with contract #DACW 61-78-C-0052 between O'Brien and Gere Engineers, Inc., Justin and Courtney Division, and the United States Army Corps of Engineers, Philadelphia District.

b. Purpose of Inspection

The purpose of this inspection is to evaluate the structural and hydraulic condition of the Packanack Lake Dam and appurtenant structures, and to determine if the dam constitutes a hazard to human life or property.

1.2 DESCRIPTION OF PROJECT

a. Description of Dam and Appurtenances

Packanack Lake Dam is a rolled earth embankment dam with a clay puddle core wall. The dam has a maximum height of about fifteen feet and is about twenty three hundred feet long. The top width of the dam is twelve feet and is surfaced with a bituminous concrete walkway. The slopes of the dam are heavily covered with brush and trees up to twelve inches in diameter. Osbourne Terrace, a paved street, is located immediately downstream of the dam.

The spillway is located at the east end of the dam and consists of a sixty feet wide ungated concrete overflow section, with three 1.5 foot wide piers spaced fifteen feet apart, supporting a bridge over the spillway. Concrete stoplog supports (one foot high) are located between the bridge piers. An eighteen inch diameter cast iron sanitary sewer pipe is exposed in the outlet channel immediately downstream of the spillway crest, and is a considerable obstruction to the spillway flow. Concrete wing walls converge rather sharply in the channel downstream of the spillway and also appear to reduce the capacity of the spillway.

The dam is equipped with a 20 inch diameter cast iron drain pipe at the low point of the reservoir near the center of the dam. The pipe is supported by concrete piers at six foot intervals from a screen chamber at the upstream side of the dam to a valve chamber at the downstream side of the dam. A 20 inch diameter pipe is connected to the downstream side of the valve chamber and discharges into Packanack Brook fifty feet downstream of the dam.

b. Location

Packanack Lake Dam is located in Wayne Township, Passaic County, New Jersey. The dam is built across Packanack Brook, about 1.5 miles from its confluence with the Pompton River.

c. Size Classification

The maximum height of the dam is about fifteen feet and the conservation storage is estimated to be 460 acre feet. Therefore, the dam is in the small size category as defined by the Recommended Guidelines for Safety Inspection of Dams.

d. Hazard Classification

Several communities are located two miles downstream of the Packanack Lake Dam, below a wide flood plain area. The ridge forming the east end wall of the reservoir runs south and then southwest from the dam. The bend of the valley wall could deflect flood discharges towards population centers. Therefore, the dam is in the high hazard category, as defined by the Recommended Guidelines for Safety Inspection of Dams.

e. Ownership

The dam is owned by the Packanack Lake Country Club and Community Association, Packanack Lake, New Jersey.

f. Purpose of Dam

The dam is used for recreation purposes.

g. Design and Construction History

The dam was designed as a rolled earth embankment with a clay puddle core by Morris R. Sherrerd for the original owner, the Irvington Hunting and Fishing Corporation. Construction of the dam took eighteen months and was completed in October of 1927 at a cost of $250,000 (See letter in appendix dated April 26, 1926, from Morris R. Sherrerd to the State of New Jersey, describing the construction of the dam).

According to the project specifications, the downstream part of the embankment was constructed of earth, sand, or gravel placed and compacted in six inch layers. The central part of the dam consists of a clay puddle core and the upstream part of the dam consists of selected clay and other fine grained soil placed and compacted in four inch layers.

h. Normal Operational Procedures

See Section 4 for information on operational procedures.

1.3 PERTINENT DATA

a. Drainage Area

The drainage area of the Packanack Lake Dam is 1.88 square miles.

b. Discharge at Dam Site

No discharge records are available for this site. According to the person in charge of operation of the dam, the maximum reservoir water surface was within two inches of the bottom of the spillway bridge girders (elevation 181.0). According to our calculations, this would produce a discharge of about 780 cubic feet per second (cfs) over the spillway. The spillway capacity with the reservoir at the dam crest elevation is about 1,400 cfs.

c. Elevation (above MSL)

Top of dam - 184.0 feet (nominal); low point - 183.4 (See Figure 3)
Maximum pool - design discharge - 184.1 feet (½PMF)
Recreation pool - 178.0 feet
Stream bed at centerline of dam - 167.0

d. Reservoir

Length of maximum pool - 6,000 feet (½PMF)
Length of recreation pool - 4,800 feet

e. Storage

Recreation pool - 460 acre feet
Design surcharge - 565 acre feet (½PMF)
Top of Dam - 554 acre feet

f. Reservoir Surface

Top of dam - 95.7 acres
Maximum pool - 96.2 acres (½PMF)
Recreation pool - 84 acres
Spillway crest - 84 acres

g. Dam

Type - rolled earth embankment
Length - 2,300 feet
Height - 15 feet (maximum)
Freeboard between normal reservoir and the top of the dam - 6 feet
Top width - 12 feet
Side slopes - 2:1; top five feet $1\frac{1}{2}$:1 (Horizontal:Vertical)
Zoning - selected earth fill with clay core
Impervious core - clay puddle
Grout curtain - none recorded

SECTION 2- ENGINEERING DATA

2.1 DESIGN

The information available for review for the Packanack Lake Dam included:

1) Contract Description of Work and Specifications for the Construction of Packanack Dam, Wayne, N.J., April 28, 1926.

2) Drawings titled "Proposed Packanack Dam of the Irvington Hunting and Fishing Corporation", April 1, 1926. These drawings consist of Elevations and Sections of the dam, drain pipe, bridge, and spillway. (See Figures 2 and 4 in the appendix).

3) Wayne Township Topographic Maps, Numbers 80 and 81, with sanitary sewer line locations.

4) Topographic map titled "Existing Topography of Bridge and Approaches of Osborne Terrace", August 1970, (See Figure 3 in the appendix).

5) Wayne Township Street Map. (See Figure 1A in the appendix).

6) Subdivision maps of the areas adjoining Packanack Lake. (See Figure 5 in the appendix).

2.2 CONSTRUCTION

The information regarding the dam construction included photographs and monthly progress reports. The progress reports list only percent complete and do not include a log of events during the construciton.

2.3 OPERATION

See Section 4.

2.4 EVALUATION

The Engineering Data reviewed indicates that the embankment was carefully constructed. However, the lack of detailed construction records makes it impossible to make a complete evaluation. Additional information required for a complete evaluation, would include:

1) As built drawings of the dam.

2) Material properties of the earth embankment, the clay puddle core wall, and the foundation.

3) A record of piezometric levels in the embankment, immediately downstream of the embankment and at each abutment.

SECTION 3 - VISUAL INSPECTION

3.1 FINDINGS

a. General

The visual inspection of the Packanack Lake Dam took place on January 12, 1978. A previous inspection was made on December 2, 1975 for the New Jersey Department of Environmental Protection by the consulting firm of Pandullo Quirk Associates of Wayne, New Jersey. An inspection was also made on July 14, 1969 by George L. Sullivan, Consulting Engineer. Both of these reports are included in the appendix.

b. Dam

The dam embankment appears to be in fair condition. Some embankment erosion has occurred adjacent to the spillway. The dam slopes are heavily covered with heavy brush and trees up to twelve inches in diameter. No evidence of significant embankment seepage or settlement was observed, although some eroded areas and minor settlement were noted. A 15 inch diameter sanitary sewer line, an eight inch water line, a gas main, and a telephone conduit are located in the downstream portion of the embankment or under the roadway immediately downstream of the embankment.

c. Appurtenant Structures

A considerable amount of concrete spalling and deterioration on and around the spillway, as well as cracking of the concrete piers supporting the bridge over the spillway was observed. The downstream end of the spillway has been undermined, and part of the spillway sill is broken away. An 18 inch diameter sanitary sewer line is located in the outlet channel just downstream of the spillway. The sewer line affects the discharge capacity of the spillway, and contributes to the erosion taking place downstream of the spillway by directing the flow toward the stream bed..

d. Reservoir Area

According to the dam owners, siltation of the reservoir is controlled by a regular dredging program.

e. Downstream Channel

Some erosion of the downstream channel is occurring.

3.2 EVALUATION

The main subjects of concern to the inspection team were:

a. The reduced capacity of the spillway caused by the bridge piers, stoplog supports and 18 inch sanitary sewer line.

b. The structural condition of the bridge piers.

c. The heavy tree & brush growth on the dam slopes.

d. The presence of the utility lines in the embankment.

Further discussion and evaluation of these subjects are covered in Section 7.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 PROCEDURES

Operational procedures were not observed by the inspection team. The description of the procedures included herein is the result of discussions with Mr. George Luckman, Manager of the Packanack Lake Country Club and Community Association. The dam and reservoir are owned and maintained by the club.

During normal conditions, the water surface elevation of the reservoir is at the spillway crest.

Procedures concerning operation of the reservoir consist of:

1) A regular program of dredging along the shoreline to remove silt deposited at storm drainage outfalls.

2) Occasional reservoir drawdown for inspection and maintenance purposes.

3) Aeration of the reservoir for improvement of water quality.

4) In addition, a recreation pond is separated from the main part of the reservoir by a rubber membrane supported by floats. Well water is pumped into the pond area to provide water of a lower temperature for swimming.

4.2 MAINTENANCE OF DAM

Complete inspection and maintenance of the upstream side of the dam can be accomplished through drawdown of the reservoir through the twenty inch diameter cast iron drain pipe or with the help of a diver. With the starting reservoir water surface at the spillway crest and no inflow, it is estimated that ten days would be required to drain the reservoir. Drawdown calculations are included in the appendix to this report. In February, 1977, the reservoir level was drawn down 4.5 feet due to potential ice damage. The last time the reservoir was completely drawn down was in 1953 for inspection of the upstream face of the dam and the drain pipe intake. The dam owner estimates that the reservoir can be drawn down at a maximum rate of five to six inches per day.

4.3 MAINTENANCE OF OPERATING FACILITIES

The bridge and spillway at Packanack Lake Dam are maintained by Passaic County, New Jersey, and the road downstream of the dam (Osborne Terrace) is maintained by Wayne Township.

4.4 DESCRIPTION OF ANY WARNING SYSTEM IN EFFECT

The Superintendent of Public Works and Police Captain of Wayne Township monitor the dam during floods and contact Civil Defense for evacuation.

4.5 EVALUATION

The drawdown facilities for the reservoir will not permit a rapid drawdown in the event of an emergency. On the basis of the visual examination and the discussions with the club manager, other operational procedures for the dam appear to be adequate.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES

a. Design Data

The Probable Maximum Flood (PMF) hydrograph was calculated from the Probable Maximum Precipitation using standard reduction factors. The Soil Conservation Service curvilinear unit hydrograph was developed for the drainage basin and used as a basis for constructing the PMF hydrograph. Both the PMF and one half of the PMF were routed through the reservoir. Peak inflow discharges to the reservoir for the PMF and one half of the PMF are 11,676 cfs and 5,836 cfs respectively. These hydrographs were routed through the reservoir by the modified-Puls method, utilizing computer program HEC-1. The peak outflow discharges for the PMF and one half of the PMF were 10,986 cfs and 2,468 cfs respectively. Both of these discharge hydrographs resulted in overtopping the dam.

b. Experience Data.

No records of reservoir stage or spillway discharge are maintained for this site. However, according to the club manager, the maximum reservoir elevation was about two inches below the spillway bridge girders (elevation 181.0 MSL). This corresponds to a discharge of approximately 780 cfs, according to our calculations.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations and Data Review

The original concrete spillway section has been covered with a concrete slab. A considerable amount of erosion has occurred at the downstream edge of this slab. Cracks and erosion of the concrete piers supporting the bridge over the spillway were also observed. The concrete retaining wall downstream of the spillway is in poor condition and some repairs have been made to this wall. This deterioration of the retaining wall appears to be more of an aesthetic problem than a structural one. However, the restriction in flow area produced by the converging wall appears to reduce the capacity of the spillway.

The earth embankment appeared to be in fair condition. No evidence of seepage or unusual settlement was observed except for the sloughing near the spillway described below. The embankment slopes are covered with heavy brush and small to medium sized trees (up to 12 inches in diameter, most on the order of 6 inches in diameter and smaller). This is a potential problem since the roots could provide a path of seepage through the embankment. However, there was no evidence of seepage through the embankment at the time of the inspection. A number of areas along the dam showed evidence of localized erosion. These did not appear to endanger the embankment although these areas should be watched and repaired by filling with suitable earth material. The upstream face of dam did not appear to be adequately riprapped. The main protection for the upstream slope against erosion and wave action appeared to be the tree and brush cover. Erosion and sloughing of the upstream face of the embankment has been taking place near the spillway. This area should be repaired and protected against further damage, since continued movement of the earth and riprap will affect the safety of the embankment and could cause a blockage of the spillway.

A 15 inch diameter sewer line (believed to be cast iron) runs longitudinally through the embankment near the downstream toe. Five manholes and a concrete overflow chamber are located in the downstream embankment. A 15 inch diameter pipe leads out of the overflow chamber and a 24 inch diameter pipe leads out of one of the manholes. Both of these pipes lead downstream of the embankment below the ground. In addition to the sewer line, an eight inch diameter water line and a telephone line are located at the downstream

toe of the dam. The paved surface of Osbourne Terrace road is also located just downstream from the dam.

The presence of these pipelines in the embankment is not desirable. No evidence of leakage was observed along any of these lines. The condition of these lines was not observed since they are buried in the embankment. The water main was reported to have broken downstream of the dam in 1971 and was repaired. No other problems with the pipelines were observed or reported.

The downstream face of the embankment has a 2:1 slope (horizontal to vertical) except for the upper 5 feet above the normal water level, which has a 1½:1 slope. These slopes are steeper than might be expected of an earth fill dam of this type. However, the normal head differential between the reservoir water surface and the ground downstream of the dam is only eight feet and the length of the seepage path from the normal reservoir surface to the ground surface downstream of the dam is about 44 feet. Because of the long seepage path, the low differential head from the reservoir to the toe of the dam, and since the embankment has performed satisfactorily for fifty years without showing signs of unusual settlement or seepage, the embankment slopes appear to be adequate.

The upstream slopes of the embankment are identical to the downstream slopes. It is likely therefore, that the upstream slope would be unstable under rapid drawdown conditions. However, the capacity of the drain pipe is such that it would take about ten days to empty the reservoir.

b. Seismic Stability

Packanack Lake Dam is located on the broad, relatively flat flood plain of Packanack Brook. The dam abutments lie in residual soils of Triassic basalts (left) and shales and sandstones of the Brunswick formation (right). Rock outcrops of the basalt occur downstream of the dam on the left abutment, whereas outcrops of the softer and weathered Brunswick sedimentary units are difficult to find. Shallow subsurface explorations made along the axis of the proposed dam in 1926 during the design phase indicated that the cutoff trench for the "puddled clay" core was excavated through several feet of muck and terminated in clay. No description is given as to the clay's overall character, color or origin except that a Mr. John N. Brooks, Hydraulic Engineer inspector for the State of New Jersey wrote on 10 August 1926 of the foundation that, "The clay underlying the swamp is of excellent quality and the foundation is approved". Based on its location, it is assumed that the foundation soils for the most part are residual in nature, but also may constitute ancient lake deposits. The spillway appears to have been excavated in more granular materials with the possibility

that some of the structure may be resting on basalt bedrock. The geologic map of New Jersey, Atlas Sheet No. 40, does not indicate any faulting or unusual geologic conditions existing in the lake area or near the dam structure. The dam is located in seismic risk zone 1 and has experienced minor seismic loading from earthquake activity generated in recent years along the Ramapo fault trending northeast-southwest within 4 to 5 miles of the dam. No effects of this activity have been noted in the dam or appurtenant structures. On the basis of the limited scope of this investigation static stability conditions appear to be satisfactory for the normal reservoir elevation and additional seismic investigations do not appear to be warranted.

SECTION 7 - ASSESSMENTS/REMEDIAL MEASURES

7.1 DAM ASSESSMENT

On the basis of the Phase I visual examination, the earth embankment of Packanack Lake Dam appears to be adequate for normal reservoir elevations. However, the slumping of the embankment near the spillway, as well as the other areas of local erosion, should be repaired. Trees growing in the embankment should be replaced by a more suitable type of vegetation to eliminate the increased seepage potential caused by the tree roots. The tree stumps should be destroyed by copper sulfate treatment and the voids resulting from removal filled with suitable material. The condition of the sewer line in the embankment should be determined by televising or other means. Consideration should be given to the removal of all utility lines from the embankment.

On the basis of the visual examination and the hydrologic and hydraulic calculations, improvement of the spillway appears to be warranted. The construction of a wall on the crest of the dam could be considered to provide protection against embankment overtopping. The sewer line in the outlet channel downstream of the spillway should be relocated and the outlet channel improved. The bridge piers and girders should be investigated in detail since a failure of the bridge could block the spillway. Since the spillway capacity is not sufficient to pass ½ of the PMF without overtopping the dam for normal reservoir conditions, the flashboard supports should be removed to prevent the raising of the normal reservoir level.

The 20 inch cast iron drain pipe through the embankment is supported on concrete piers resting on the clay foundation. The piers may be subject to differential settlement which could produce excessive stresses in the pipe and/or voids in the earth materials around the pipe. Since the control valve for the pipe is located in the downstream slope of the embankment, the pipe is always subject to the hydrostatic pressure produced by the reservoir and it is not possible to empty the pipe without draining the reservoir. This makes it impossible to empty the pipe rapidly in the event of a failure of or a leak in the pipe. Failure of or a leak in the pipe while under pressure could lead to a rapid progressive internal erosion failure of the embankment.

The improvements suggested should be accomplished as soon as possible. Additional investigations may be necessary in connection with the adequacy of the spillway since the calculations have shown that the spillway capacity does not meet the requirements of the Recommended Guidelines for Safety Inspection of Dams.

7.2 REMEDIAL MEASURES

Replacement of the trees with vetch or other ground covers normally used in dam embankments would be acceptable for the downstream face of the dam. On the upstream face, additional riprap may be required after removal of the trees to protect against erosion.

Removal of the bridge piers and stoplog supports and replacement of the multiple span bridge with a single span would improve the capacity of the spillway and reduce the possibility of blockage by debris during a flood. Removal of the sudden constriction in the outlet channel downstream of the spillway would probably also improve the spillway capacity.

The construction of a concrete wing wall at the upstream junction of the embankment and the spillway on the reservoir side may be required to prevent erosion of embankment in this area and subsequent blockage of the spillway. An alternate solution would be the placement of heavy riprap in this area.

Installation of a valve at the upstream end of the drain pipe would make it possible to prevent full reservoir hydrostatic pressure in the pipe and would also make it possible to empty the pipe rapidly in the event of a failure of or a leak in the pipe.

FIGURES

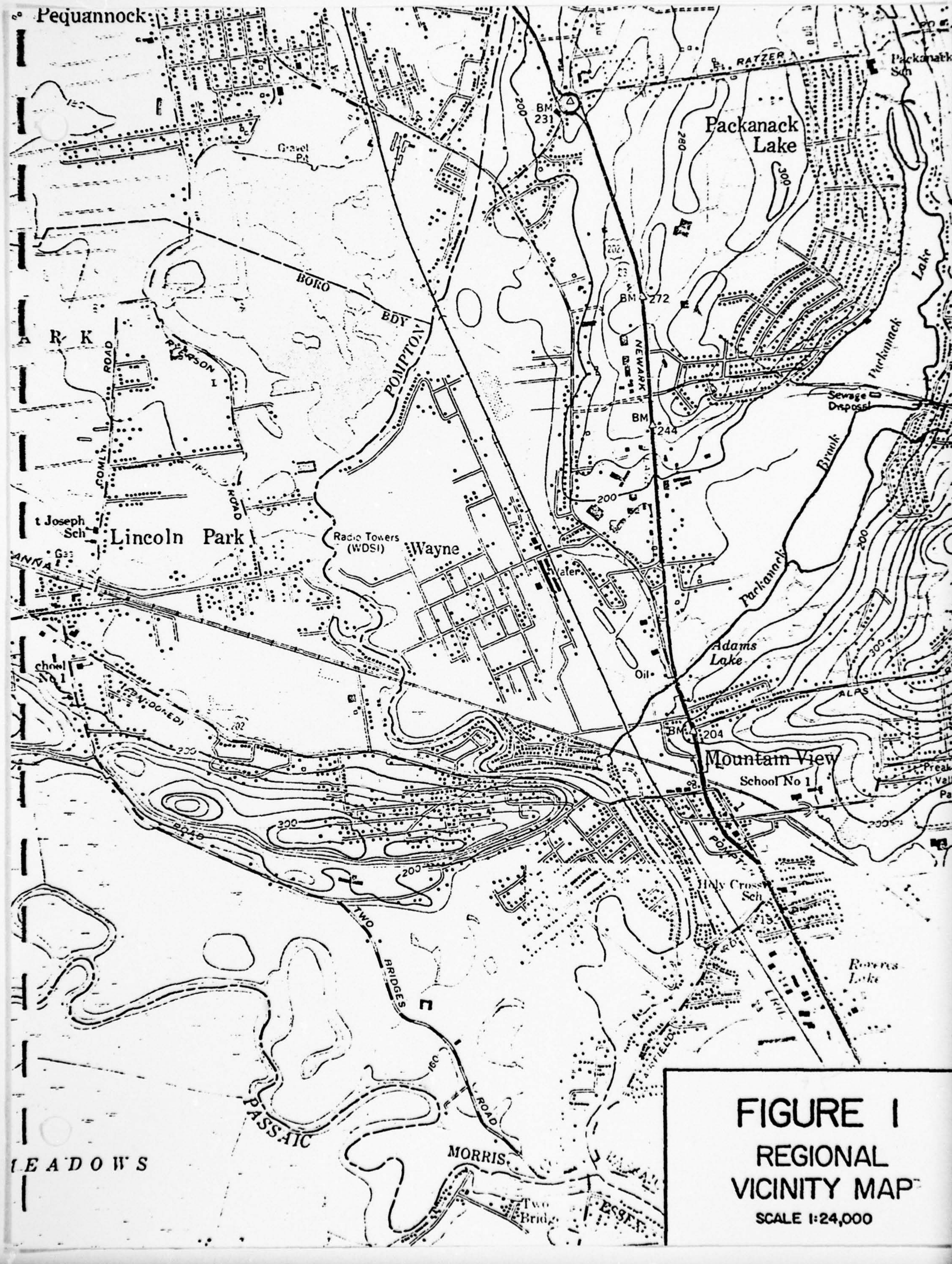
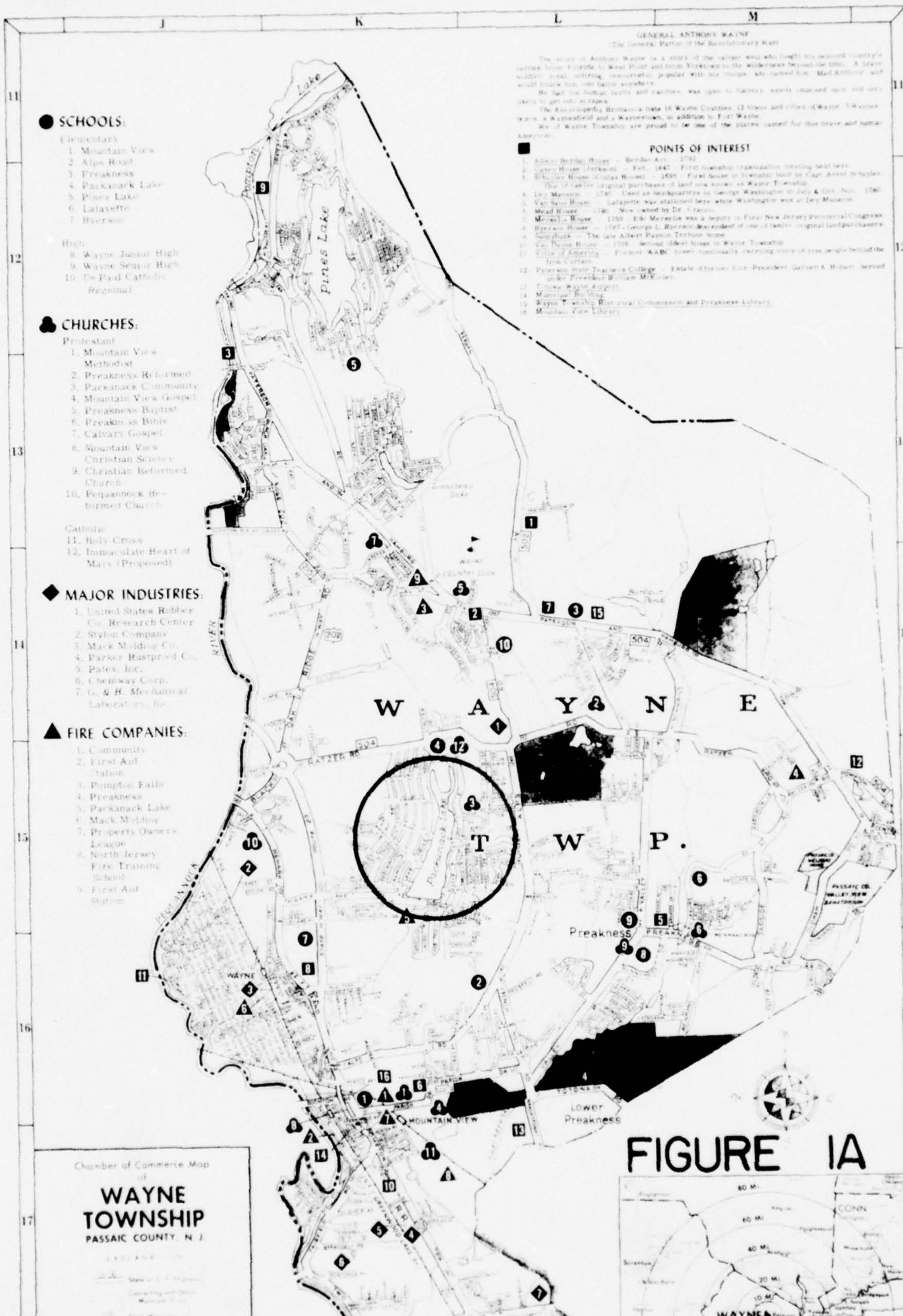


FIGURE 1

REGIONAL VICINITY MAP

SCALE 1:24,000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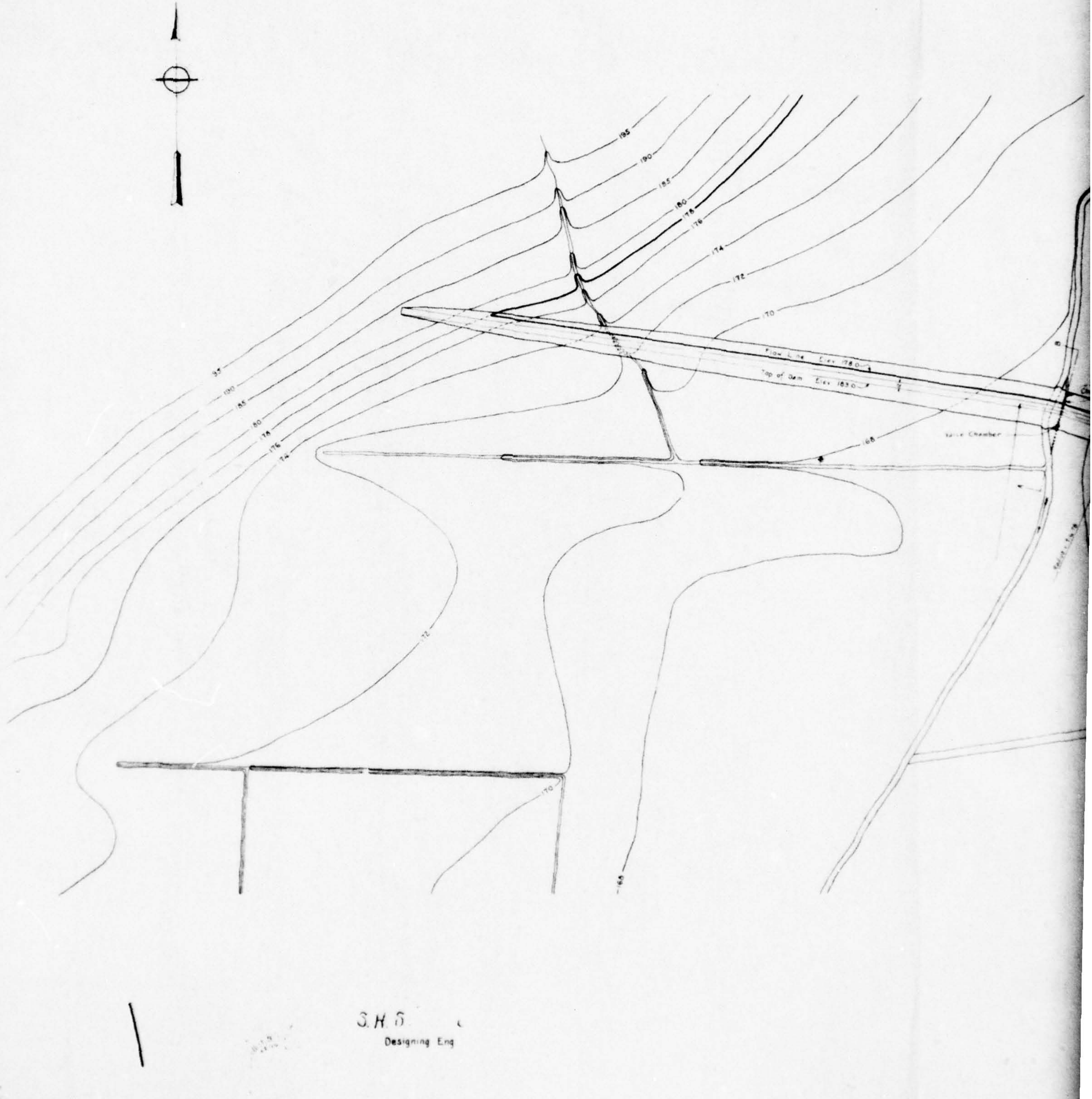
1. Mountain View
2. Alps Road
3. Preakness
4. Packanack Lake
5. Pines Lake
6. Lafayette
7. Ryerson
High
8. Wayne Junior High
9. Wayne Senior High
10. De Paul Catholic Regional
CHURCHES:
Protestant
1. Mountain View Methodist
2. Preakness Reformed
3. Packanack Community
4. Mountain View Gospel
5. Preakness Baptist
6. Preakness Bible
7. Calvary Gospel
8. Mountain View Christian Science
9. Christian Reformed Church
10. Pequannock Reformed Church
Catholic
11. Holy Cross
12. Immaculate Heart of Mary (Proposed)
MAJOR INDUSTRIES:
1. United States Rubber Co. Research Center
2. Stylon Company
3. Mack Molding Co.
4. Parker Rustproof Co.
5. Patex, Inc.
6. Chemway Corp.
7. G. & H. Mechanical Laboratory, Inc.
FIRE COMPANIES:
1. Community
2. First Aid Station
3. Pompton Falls
4. Preakness
5. Packanack Lake
6. Mack Molding
7. Property Owners League
8. North Jersey Fire Training School
9. First Aid Station
W A Y N E
T W P.
Pines Lake
Preakness
MOUNTAIN VIEW
Lower Preakness
Chamber of Commerce Map of
WAYNE TOWNSHIP
PASSAIC COUNTY, N.J.
ADDITIONAL COPIES 25¢ each
WAYNE
NEW YORK
PHILADELPHIA
CONN.
N.J.

FIGURE 1A



S.H.S.

Designing Eng

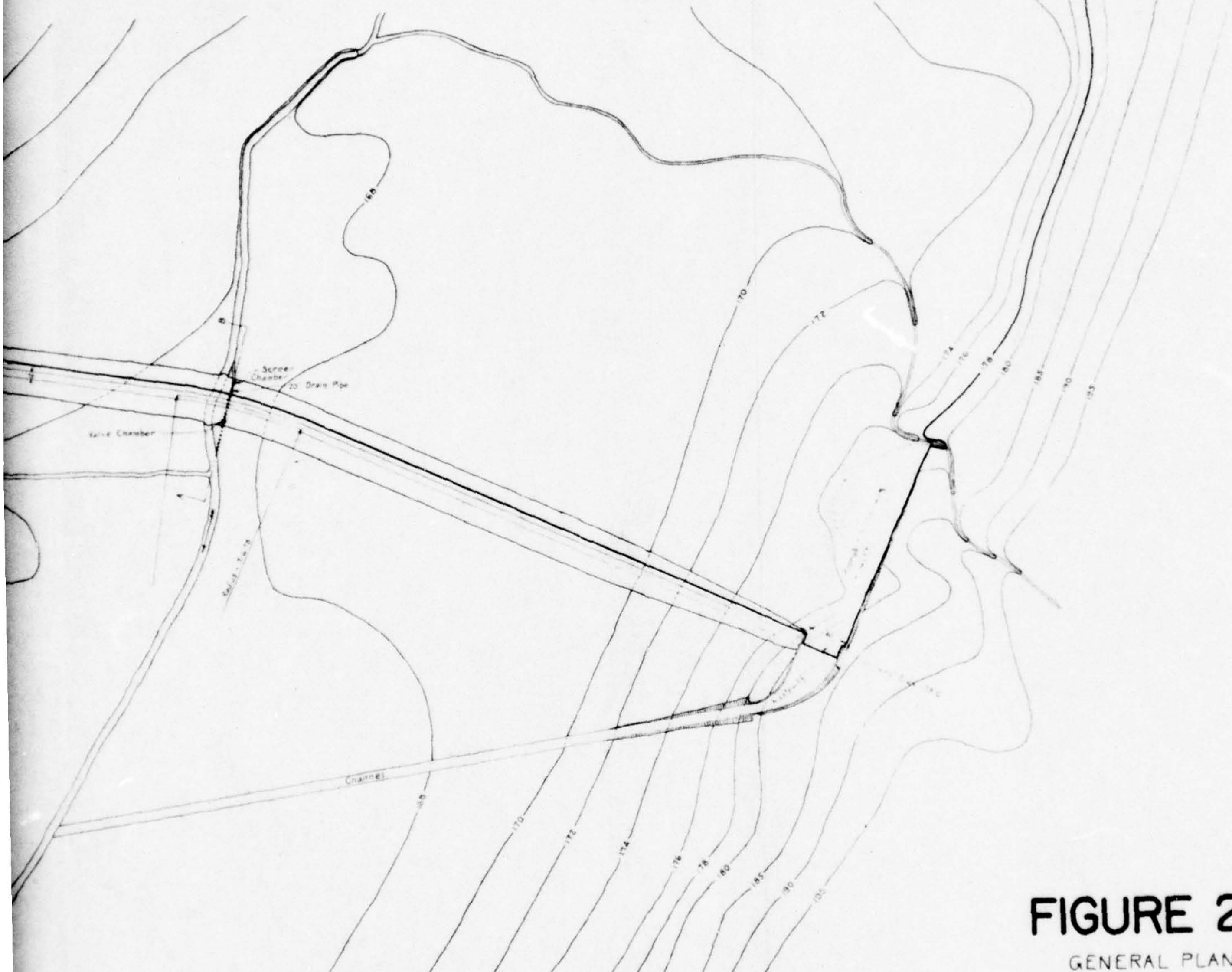
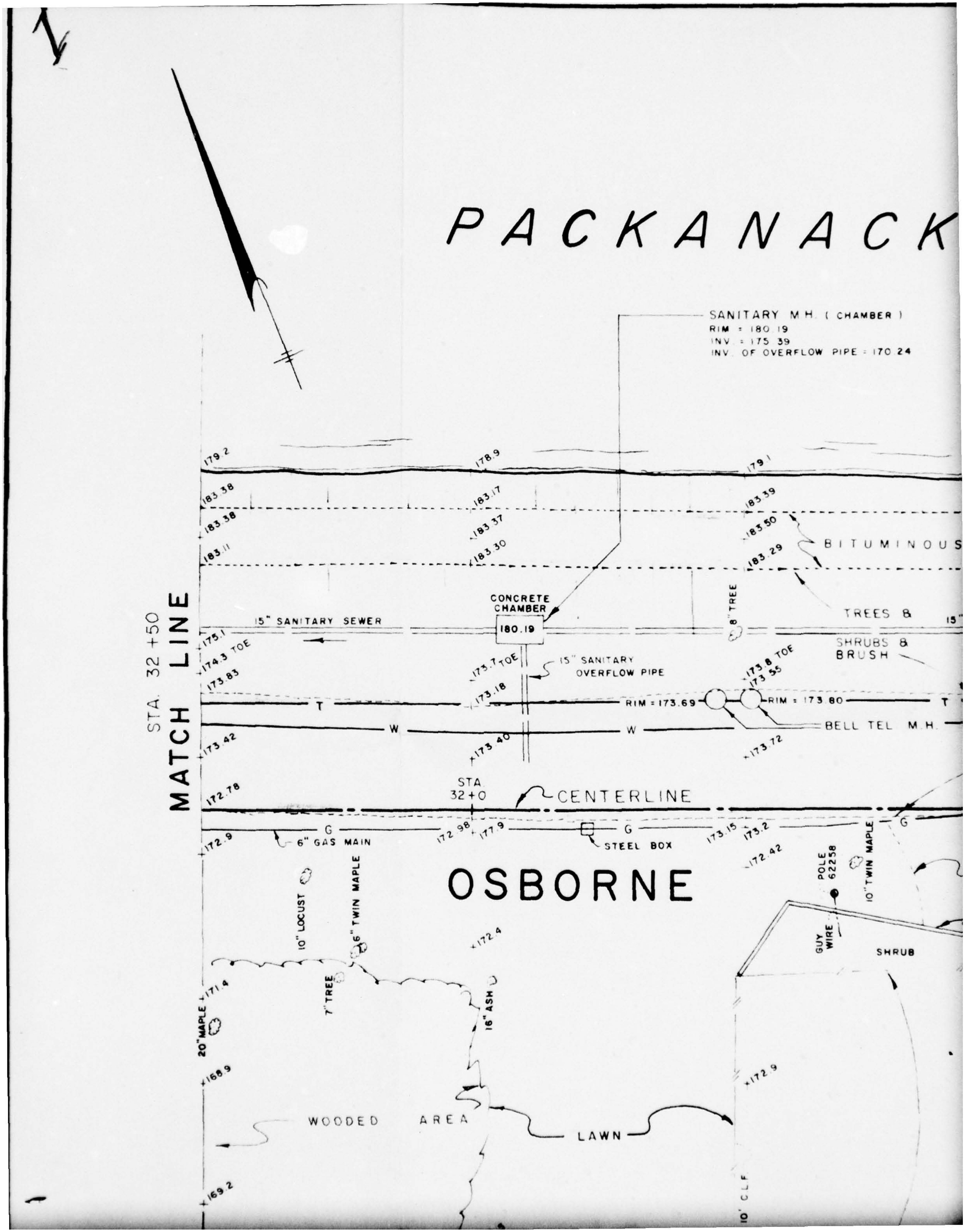


FIGURE 2

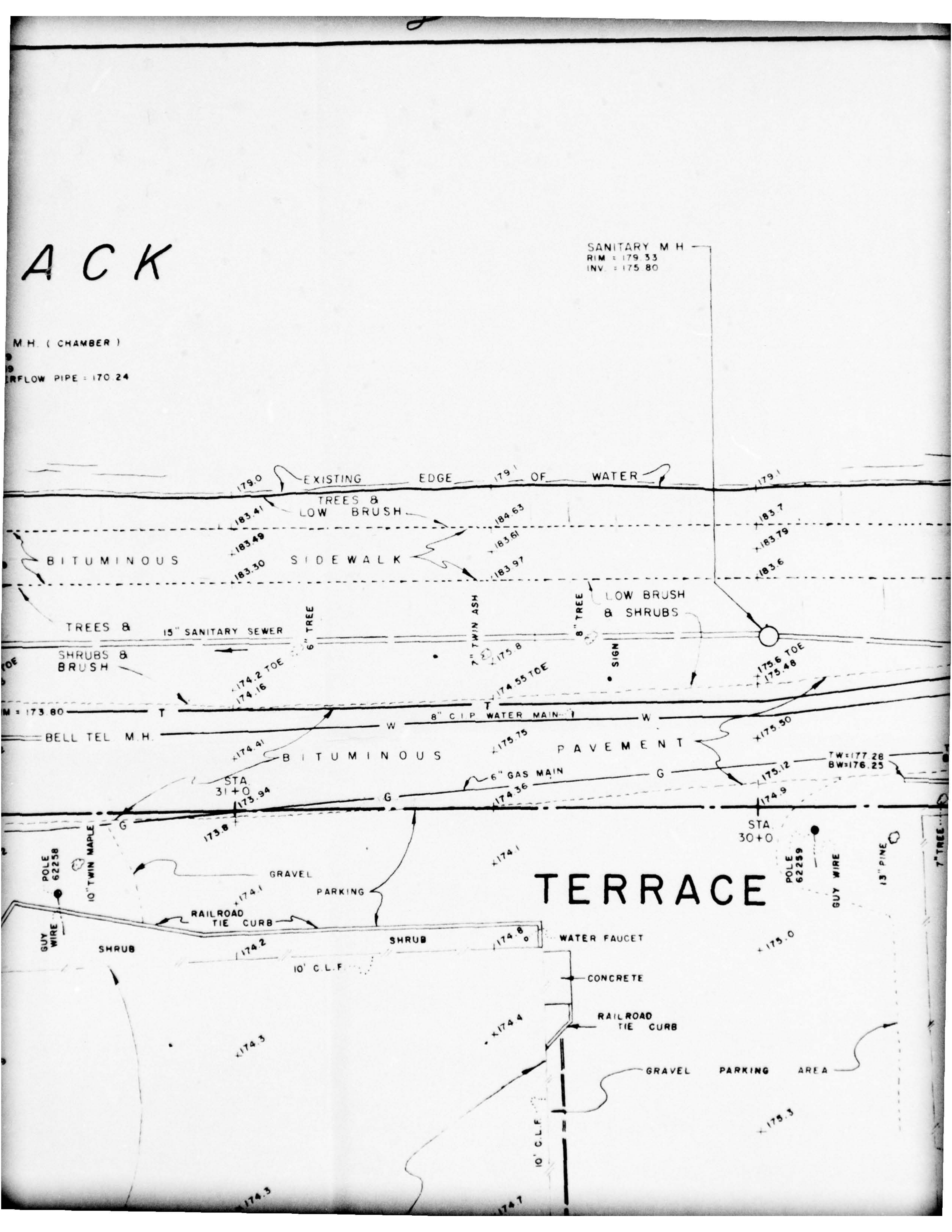
GENERAL PLAN
PROPOSED
PACKANAC LAKE
OF THE
IRVINGTON HUNTING &
FISHING CORPORATION

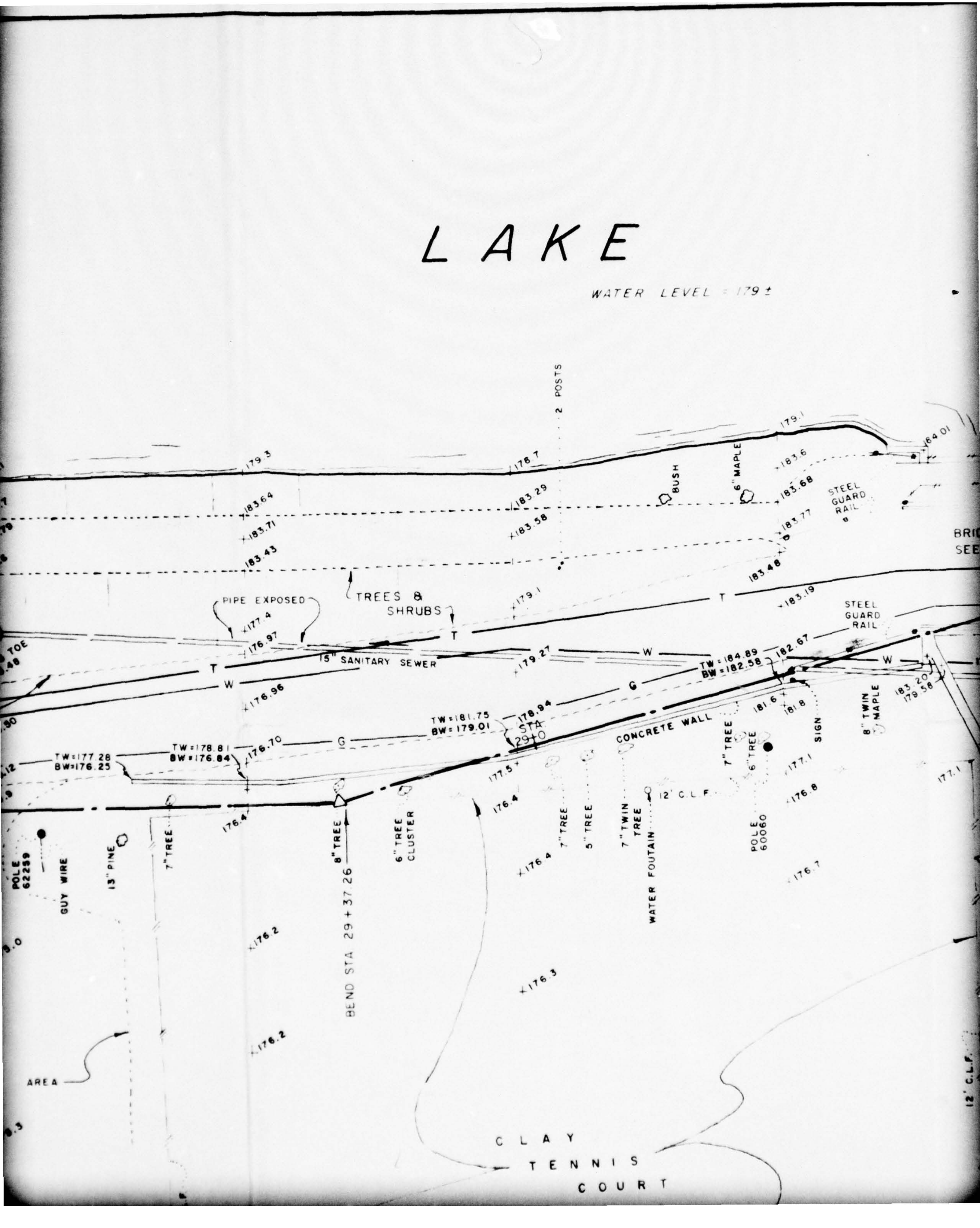
SCALE 1 in = 100 ft. APRIL 1 1926

M R Sherrerd
Consulting Eng

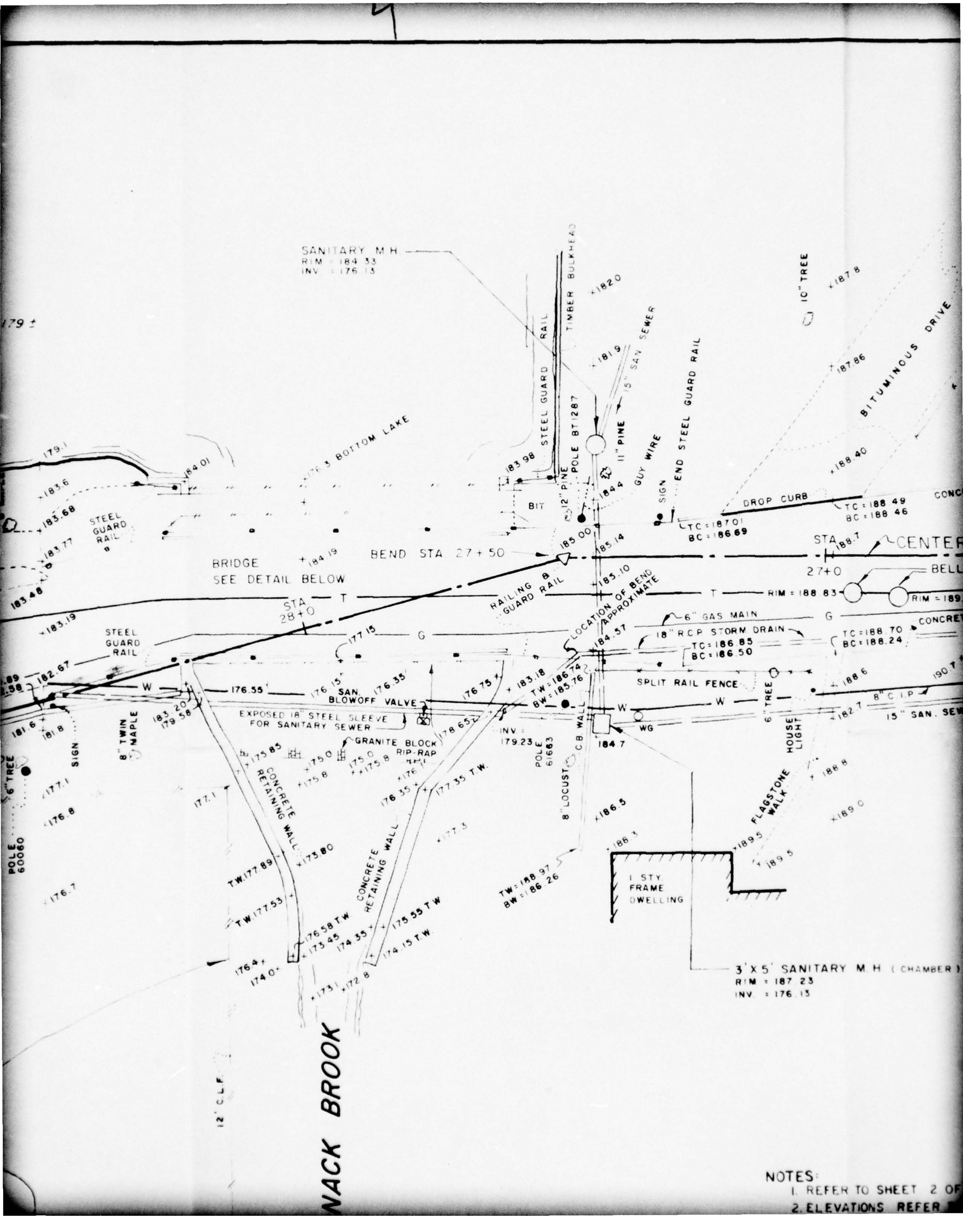


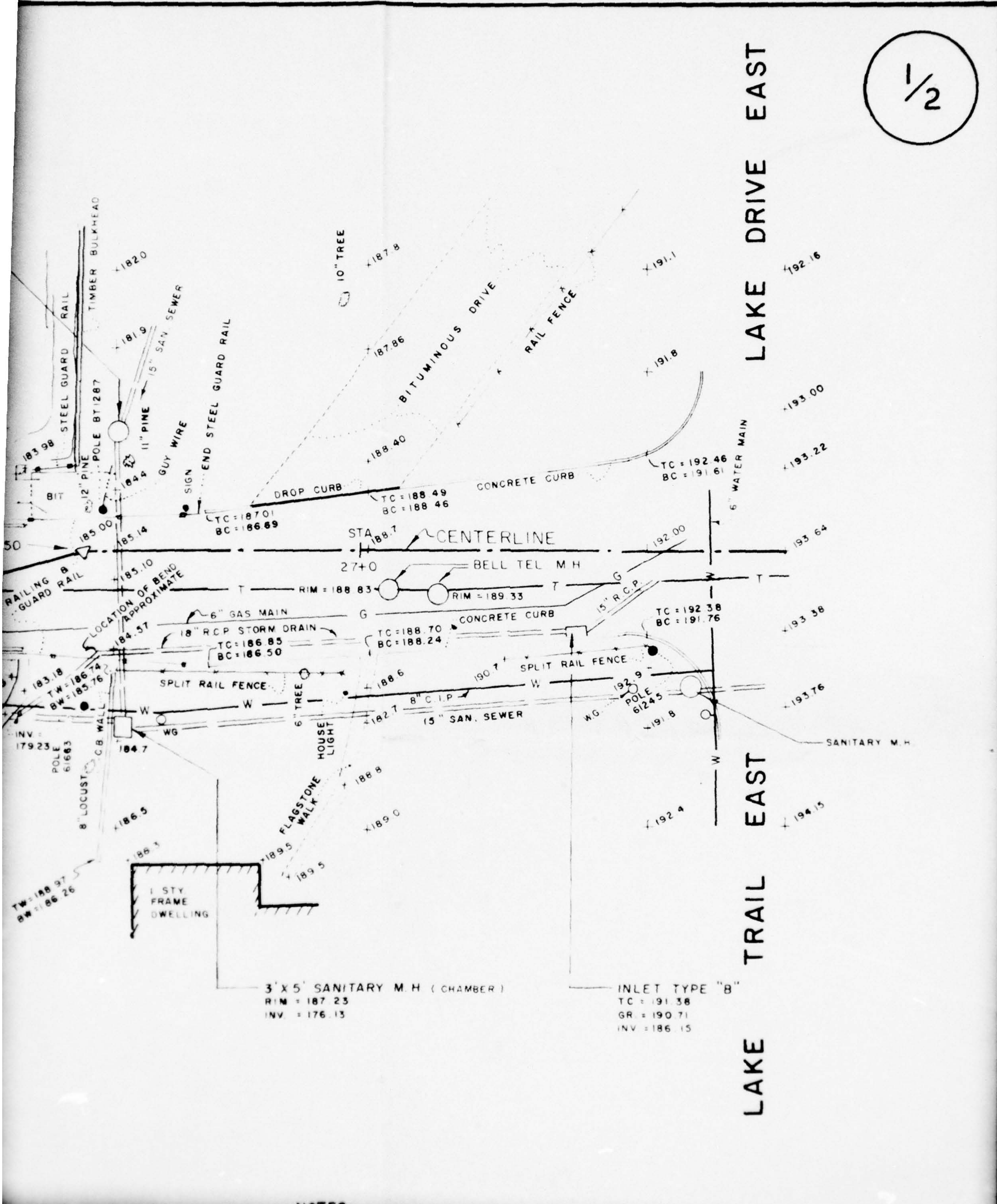
PACKANACK
SANITARY M.H. (CHAMBER)
RIM = 180.19
INV. = 175.39
INV. OF OVERFLOW PIPE = 170.24
179.2
178.9
179.1
183.38
183.17
183.39
183.38
183.37
183.50
183.11
183.30
183.29
BITUMINOUS
STA. 32 +50
MATCH LINE
CONCRETE CHAMBER
180.19
15" SANITARY SEWER
8" TREE
TREES &
SHRUBS &
BRUSH
175.1
174.3 TOE
173.7 TOE
15" SANITARY OVERFLOW PIPE
173.8 TOE
173.55
173.83
173.18
RIM = 173.69
RIM = 173.80
BELL TEL. M.H.
173.42
173.40
173.72
STA. 32+0
CENTERLINE
172.78
172.9
6" GAS MAIN
172.98
177.9
STEEL BOX
173.15
173.2
172.42
OSBORNE
10" LOCUST
6" TWIN MAPLE
POLE 62258
10" TWIN MAPLE
GUY WIRE
SHRUB
172.4
7" TREE
16" ASH
20" MAPLE
171.4
168.9
172.9
WOODED AREA
LAWN
169.2
10' C.L.F.


ACK
SANITARY M.H.
RIM = 179.33
INV. = 175.80
M.H. (CHAMBER)
RFLOW PIPE = 170.24
EXISTING EDGE OF WATER
TREES & LOW BRUSH
BITUMINOUS SIDEWALK
LOW BRUSH & SHRUBS
TREES & SHRUBS & BRUSH
15" SANITARY SEWER
6" TREE
7" TWIN ASH
8" TREE
SIGN
8" C.I.P. WATER MAIN
BELL TEL. M.H.
BITUMINOUS PAVEMENT
6" GAS MAIN
TW=177.28
BW=176.25
STA. 31+0
STA. 30+0
POLE 62258
10" TWIN MAPLE
GRAVEL PARKING
POLE 62259
GUY WIRE
13" PINE
7" TREE
TERRACE
RAILROAD TIE CURB
GUY WIRE
SHRUB
WATER FAUCET
10' C.L.F.
CONCRETE
GRAVEL PARKING AREA

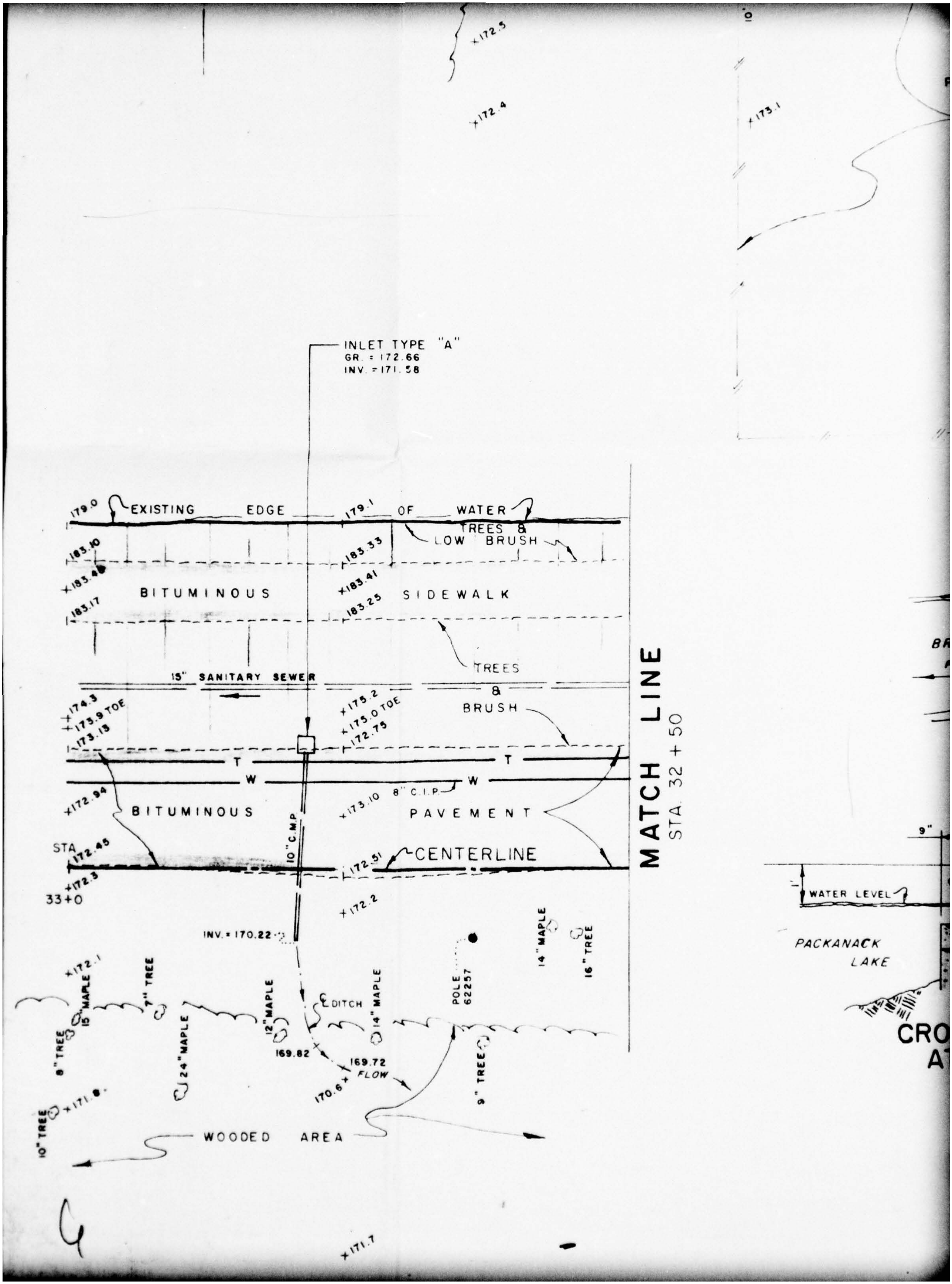


LAKE
WATER LEVEL = 179 ±
2 POSTS
179.3
178.7
179.1
184.01
183.64
183.71
183.43
183.29
183.58
BUSH
6" MAPLE
183.6
183.68
183.77
STEEL GUARD RAIL
183.48
183.19
TREES &
SHRUBS
PIPE EXPOSED
177.4
176.97
179.1
15" SANITARY SEWER
179.27
TW = 184.89
BW = 182.58
182.67
STEEL GUARD RAIL
176.96
TW = 181.75
BW = 179.01
178.94
STA 29+0
CONCRETE WALL
181.6
181.8
SIGN
8" TWIN MAPLE
183.20
179.58
TW = 177.28
BW = 176.25
TW = 178.81
BW = 176.84
176.70
177.5
7" TREE
6" TREE
177.1
177.1
176.4
176.4
12' C.L.F.
176.8
POLE 62259
GUY WIRE
13" PINE
7" TREE
8" TREE
BEND STA 29 + 37 26
6" TREE CLUSTER
7" TREE
5" TREE
7" TWIN TREE
WATER FOUTAIN
POLE 60060
176.4
176.7
176.2
176.3
176.2
AREA
12' C.L.F.
CLAY
TENNIS
COURT

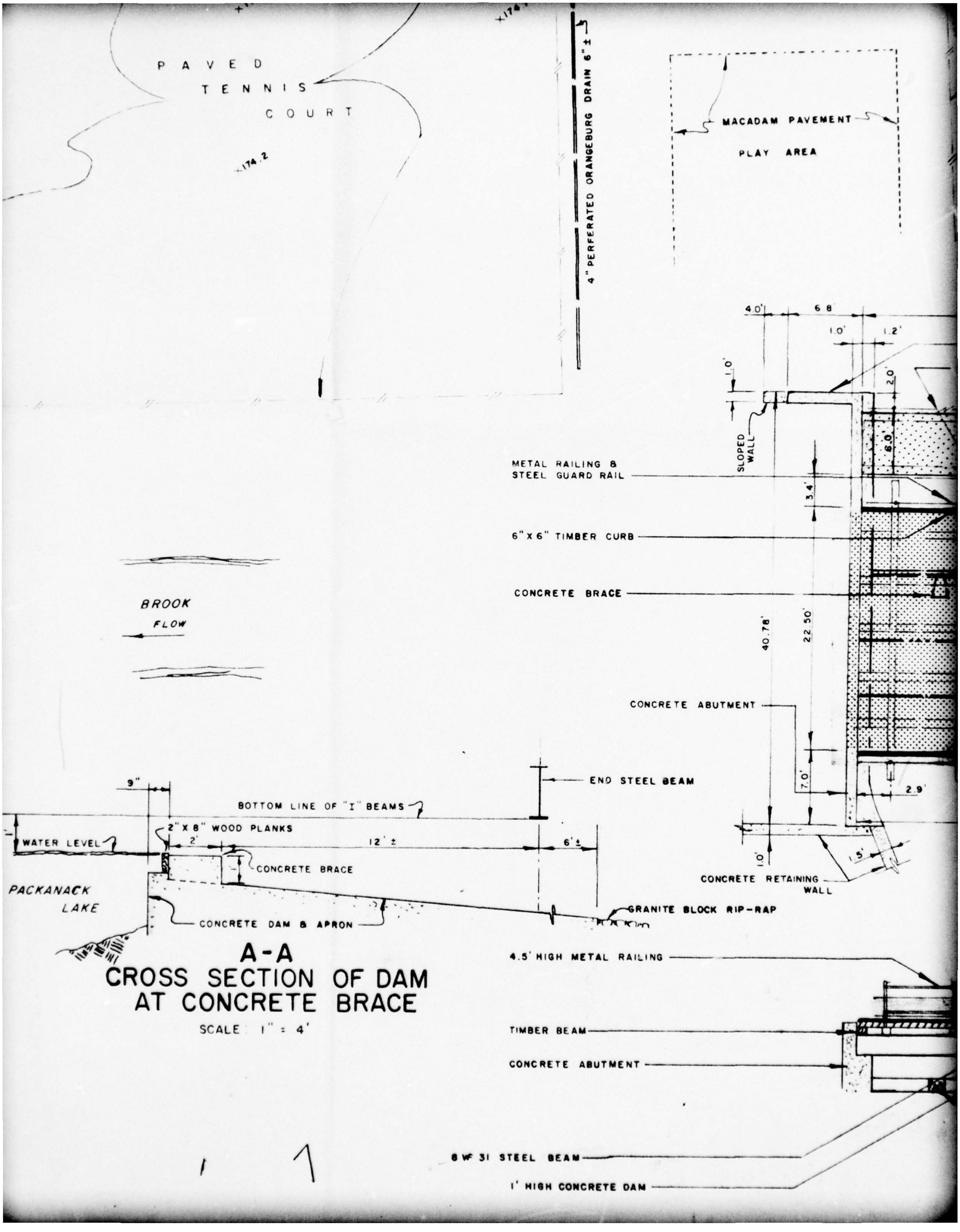

SANITARY M.H.
RIM = 184.33
INV. = 176.13
179 ±
TIMBER BULKHEAD
STEEL GUARD RAIL
15" SAN. SEWER
END STEEL GUARD RAIL
10" TREE
BITUMINOUS DRIVE
POLE BT 1287
11" PINE
12" PINE
GUY WIRE
SIGN
176.3 BOTTOM LAKE
BIT
DROP CURB
TC = 187.01
BC = 186.69
TC = 188.49
BC = 188.46
CONC
STEEL GUARD RAIL
BRIDGE
SEE DETAIL BELOW
BEND STA. 27 + 50
185.00
185.14
STA 27+0
CENTER
BELL
RAILING & GUARD RAIL
LOCATION OF BEND APPROXIMATE
RIM = 188.83
RIM = 189
STA 28+0
T
G
6" GAS MAIN
18" R.C.P. STORM DRAIN
TC = 186.85
BC = 186.50
TC = 188.70
BC = 188.24
CONCRETE
STEEL GUARD RAIL
177.15
184.57
SPLIT RAIL FENCE
W
176.55
176.15
176.35
SAN. BLOWOFF VALVE
176.75
183.18
TW = 186.74
BW = 185.76
8" C.I.P.
15" SAN. SEW
EXPOSED 18" STEEL SLEEVE FOR SANITARY SEWER
WG
6" TREE
HOUSE LIGHT
8" TWIN MAPLE
SIGN
183.20
179.58
INV. = 179.23
184.7
GRANITE BLOCK RIP-RAP
POLE 61663
C.B. WALL
8" LOCUST
6" TREE
CONCRETE RETAINING WALL
FLAGSTONE WALK
186.5
188.3
189.5
POLE 60060
CONCRETE RETAINING WALL
177.35 T.W.
TW = 188.97
BW = 186.26
1 STY. FRAME DWELLING
T.W. 177.89
T.W. 177.53
176.58 T.W.
175.55 T.W.
174.15 T.W.
3' X 5' SANITARY M.H. (CHAMBER)
RIM = 187.23
INV. = 176.13
12" C.L.F.
NACK BROOK
NOTES:
1. REFER TO SHEET 2 OF
2. ELEVATIONS REFER



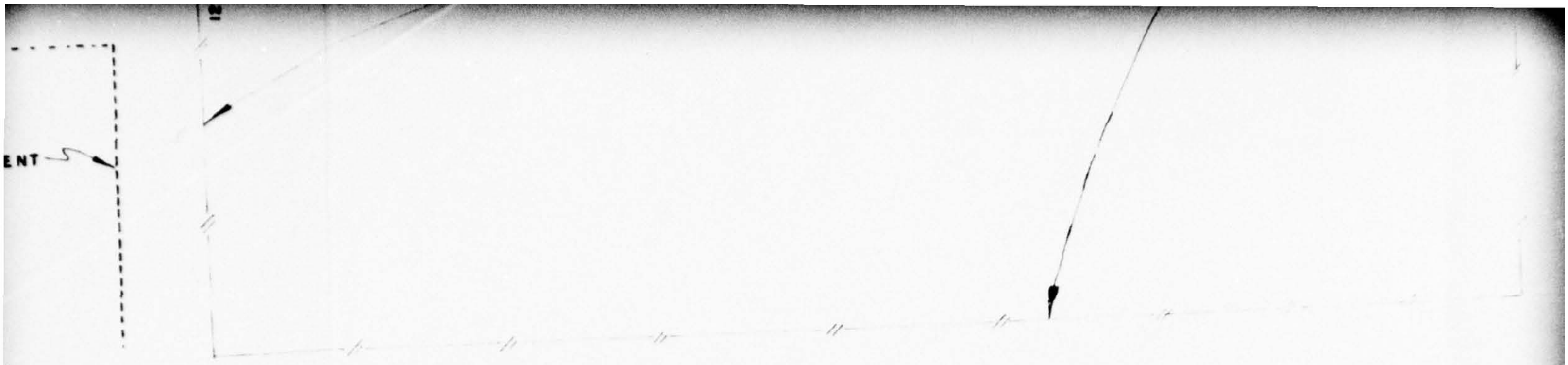
NOTES:
1. REFER TO SHEET 2 OF 2 FOR RIGHT OF WAY INFORMATION

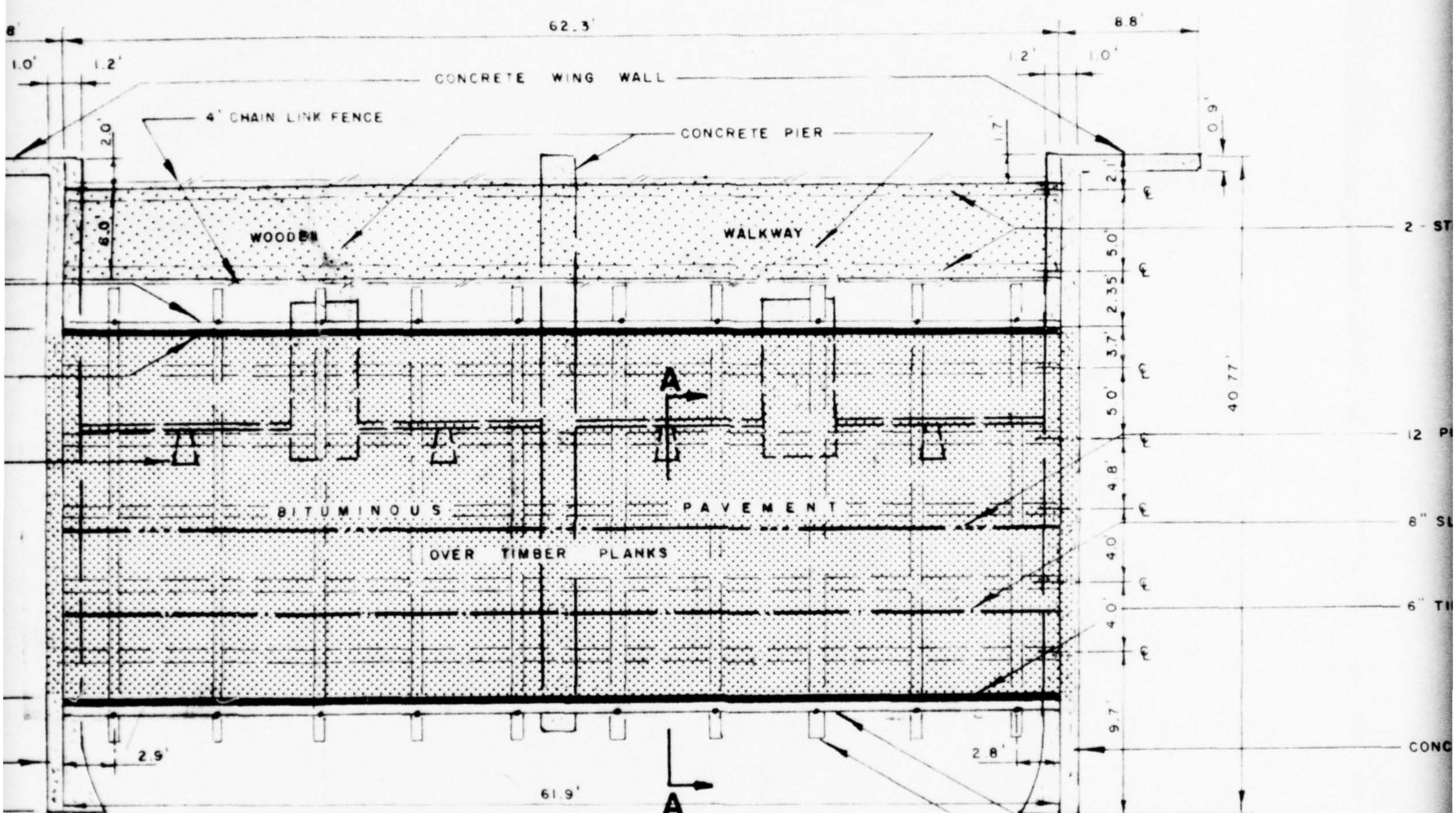


INLET TYPE "A"
GR. = 172.66
INV. = 171.58
EXISTING EDGE OF WATER
TREES & LOW BRUSH
BITUMINOUS SIDEWALK
TREES & BRUSH
15" SANITARY SEWER
BITUMINOUS PAVEMENT
8" C.I.P.
10" C.M.P.
CENTERLINE
STA 33+0
INV. = 170.22
MATCH LINE
STA. 32 + 50
POLE 62257
DITCH
FLOW
WOODED AREA
WATER LEVEL
PACKANACK LAKE
179.0
179.1
183.10
183.33
183.41
183.17
183.25
174.3
173.9 TOE
173.13
175.2
175.0 TOE
172.75
172.94
173.10
172.45
172.51
172.3
172.2
172.1
171.8
171.7
169.82
169.72
170.6
172.5
172.4
173.1
15" MAPLE
7" TREE
8" TREE
24" MAPLE
12" MAPLE
14" MAPLE
9" TREE
14" MAPLE
16" TREE
10" TREE
CRO
AT



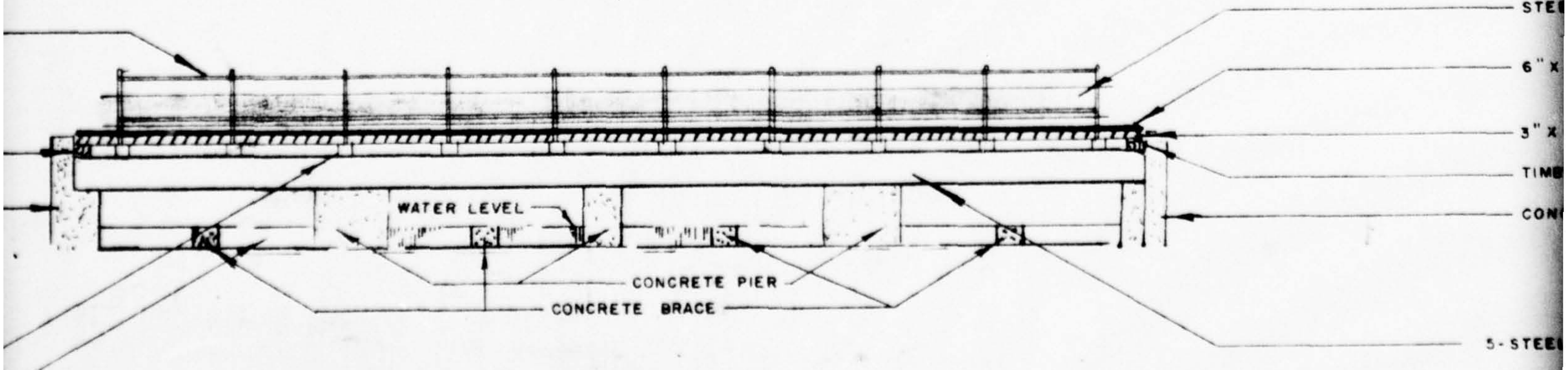
A-A
CROSS SECTION OF DAM AT CONCRETE BRACE
SCALE: 1" = 4'





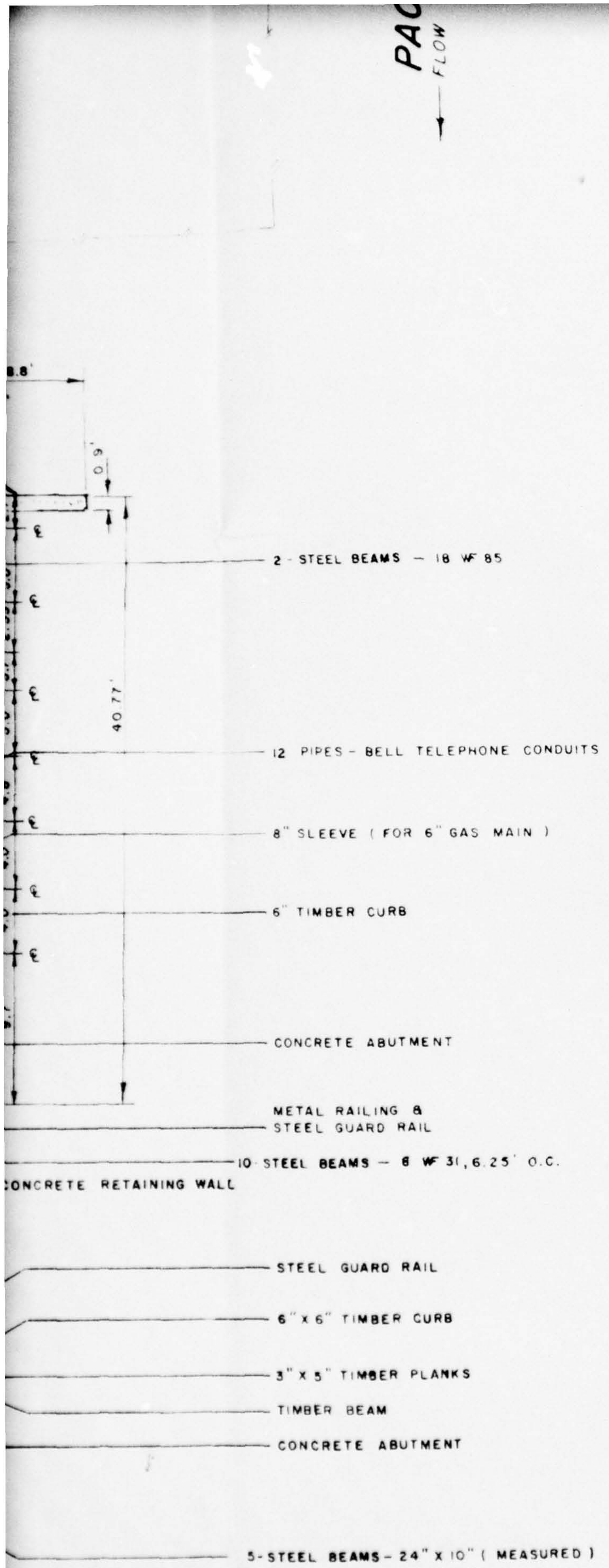
PLAN

SCALE: 1" = 10'



ELEVATION

SCALE: 1" = 10'



LEGEND:

T.C.	TOP OF CURB
B.C.	BOTTOM OF C
M.H.	MANHOLE
INV.	INVERT
— G —	GAS MAIN
— W —	WATER MAIN
— T —	TELEPHONE C
C.L.F.	CHAIN LINK F
T.W.	TOP OF WALL
B.W.	BOTTOM OF W

EXIS

BRIDGE AND

PAS

PASSAIC COU

GE

JOHN Z

ENGI

DRAWN QL
CHECK
BOOK W 81 pg 1-12
LIC NO. 9957

9

LEGEND:

T.C.	TOP OF CURB
B.C.	BOTTOM OF CURB
M.H.	MANHOLE
INV.	INVERT
—G—	GAS MAIN
—W—	WATER MAIN
—T—	TELEPHONE CONDUIT
C.L.F.	CHAIN LINK FENCE
T.W.	TOP OF WALL
B.W.	BOTTOM OF WALL

EXISTING TOPOGRAPHY

OF

BRIDGE AND APPROACHES OF OSBORNE TERRACE

IN THE

TOWNSHIP OF WAYNE

PASSAIC COUNTY, NEW JERSEY

FOR

PASSAIC COUNTY ENGINEERING DEPARTMENT

GEORGE MASON - ENGINEER

JOHN ZANETAKOS ASSOCIATES, INC.

ENGINEERING - SURVEYING - PLANNING

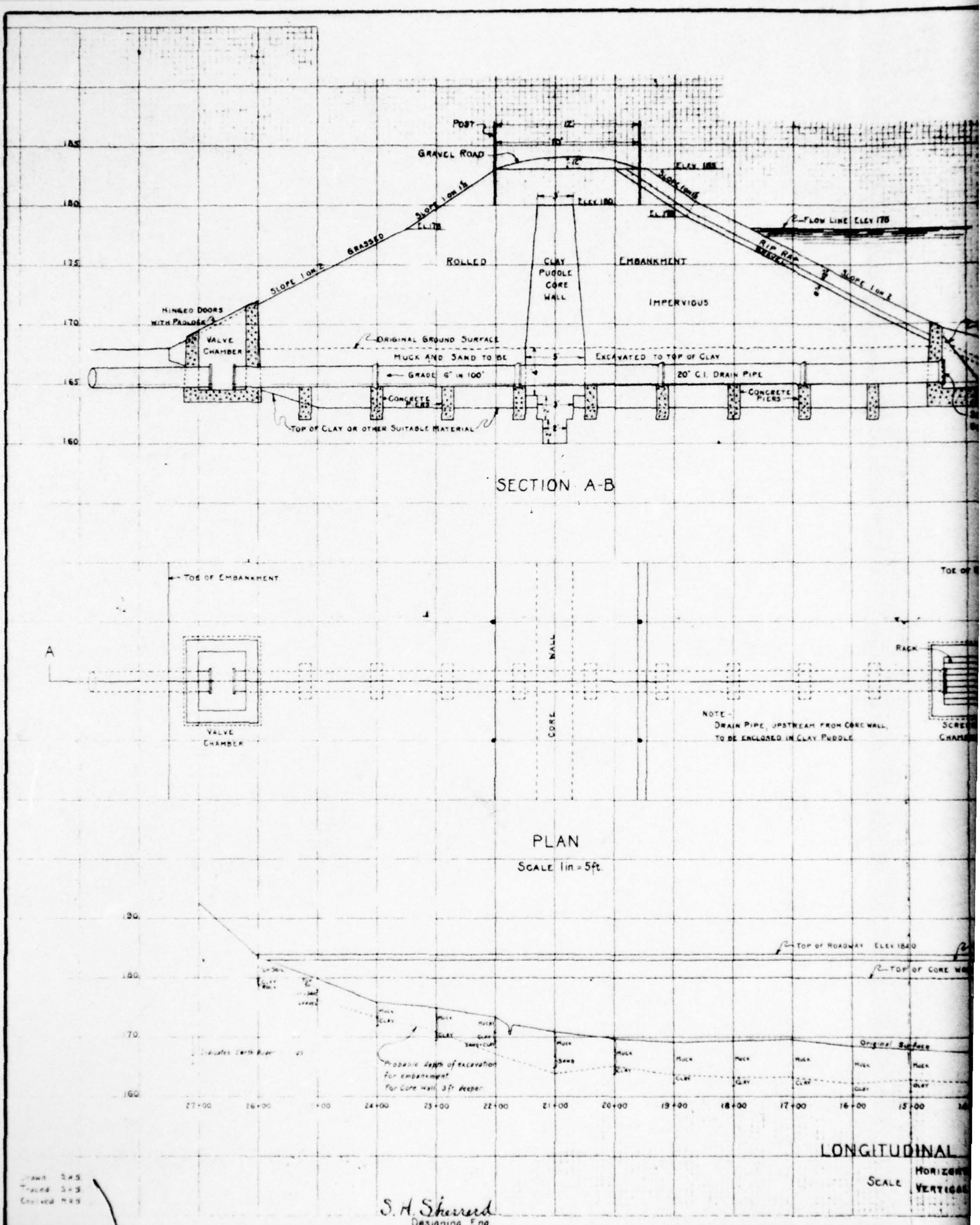
WAYNE, NEW JERSEY

DRAWN QL
CHECK
BOOK W81 pg 1-12
LIC NO. 9957

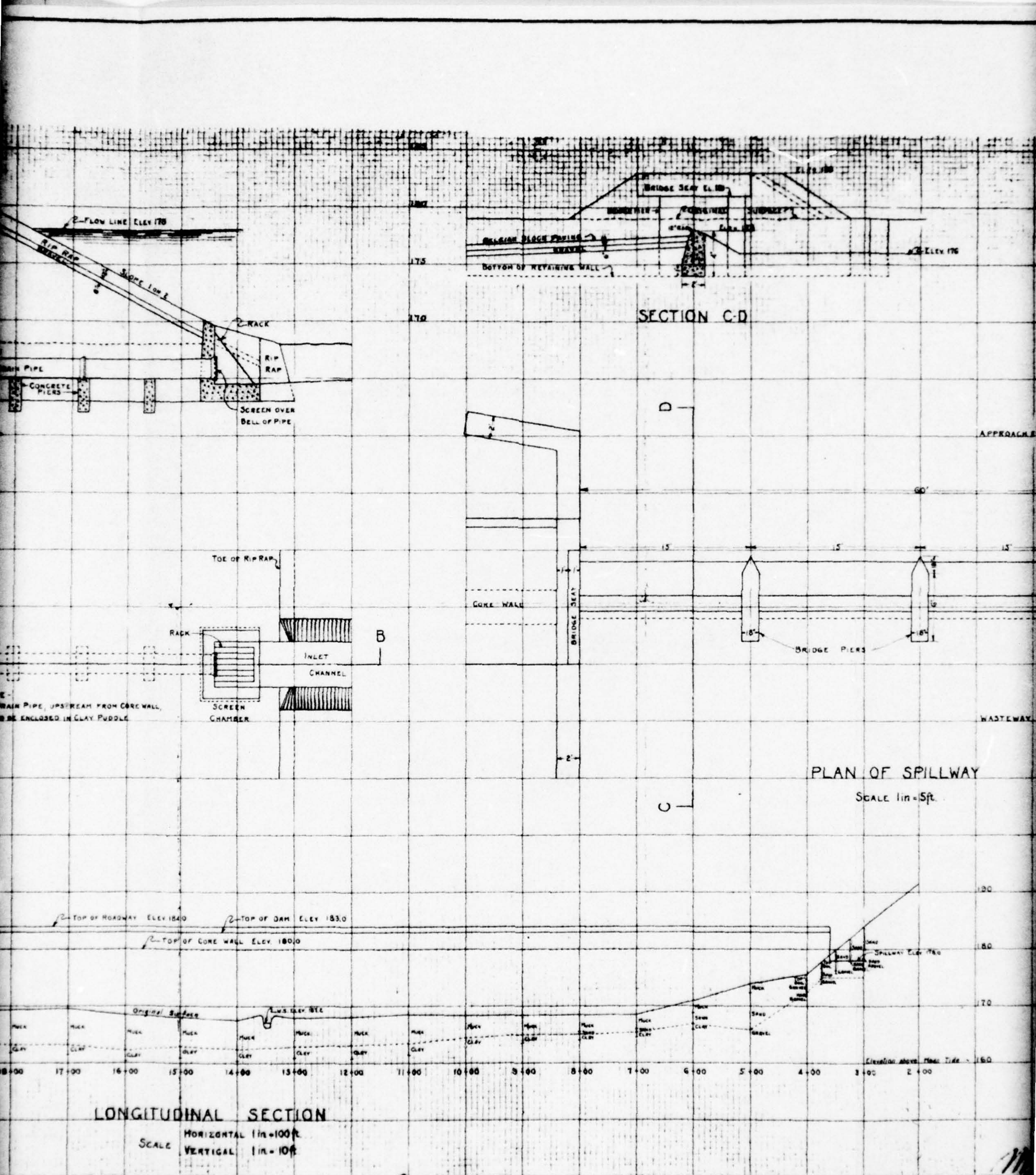
John L. Zanetakos
JOHN L. ZANETAKOS

DATE: AUGUST 1970
SCALE: 1" = 20'
SHEET 1 OF 2
JOB NO. 70-5061

10



POST
12'
10'
GRAVEL ROAD
12"
ELEV 185
SLOPE 1 ON 1½
SLOPE 1 ON 1½
ELEV 180
EL 178
EL 178
FLOW LINE ELEV 178
GRASSED
SLOPE 1 ON 2
RIP RAP
SLOPE 1 ON 2
ROLLED
CLAY PUDDLE CORE WALL
EMBANKMENT
IMPERVIOUS
HINGED DOORS WITH PADLOCK
VALVE CHAMBER
ORIGINAL GROUND SURFACE
MUCK AND SAND TO BE
EXCAVATED TO TOP OF CLAY
GRADE 6" IN 100'
20" C.I. DRAIN PIPE
CONCRETE PIERS
CONCRETE PIERS
TOP OF CLAY OR OTHER SUITABLE MATERIAL
185
180
175
170
165
160
SECTION A-B
TOE OF EMBANKMENT
A
VALVE CHAMBER
CORE WALL
RACK
NOTE:- DRAIN PIPE, UPSTREAM FROM CORE WALL, TO BE ENCLOSED IN CLAY PUDDLE
PLAN
SCALE 1in = 5ft.
TOP OF ROADWAY ELEV 184.0
TOP OF CORE WA
Original Surface
MUCK
CLAY
SAND
Probable depth of excavation for embankment For Core Wall 3ft deeper
190
180
170
160
27+00
26+00
24+00
23+00
22+00
21+00
20+00
19+00
18+00
17+00
16+00
15+00
LONGITUDINAL
SCALE HORIZONT VERTICAL
Drawn S.H.S.
Traced S.H.S.
Checked H.R.S.
S. H. Sherrard
Designing Eng



FLOW LINE ELEV 178
RIP RAP
SLOPE 1 ON 2
RACK
RIP RAP
DRAIN PIPE
CONCRETE PIERS
SCREEN OVER BELL OF PIPE
175
170
BRIDGE SEAT
BELGIAN BLOCK PAVING
GRAVEL
BOTTOM OF RETAINING WALL
ELEV 176
SECTION C-D
D
APPROACH
60'
15'
15'
15'
TOE OF RIP RAP
CORE WALL
BRIDGE SEAT
18"
18"
BRIDGE PIERS
RACK
B
INLET
CHANNEL
SCREEN
CHAMBER
DRAIN PIPE, UPSTREAM FROM CORE WALL, TO BE ENCLOSED IN CLAY PUDDLE
2'
WASTEWAY
PLAN OF SPILLWAY
Scale 1in = 5ft.
C
190
180
170
160
TOP OF ROADWAY ELEV 184.0
TOP OF DAM ELEV 183.0
TOP OF CORE WALL ELEV 180.0
SPILLWAY ELEV 178.0
Original Surface
MUCK
CLAY
SAND
GRAVEL
Elevation above Mean Tide
18+00
17+00
16+00
15+00
14+00
13+00
12+00
11+00
10+00
9+00
8+00
7+00
6+00
5+00
4+00
3+00
2+00
LONGITUDINAL SECTION
Scale
HORIZONTAL 1in = 100ft
VERTICAL 1in = 10ft

2

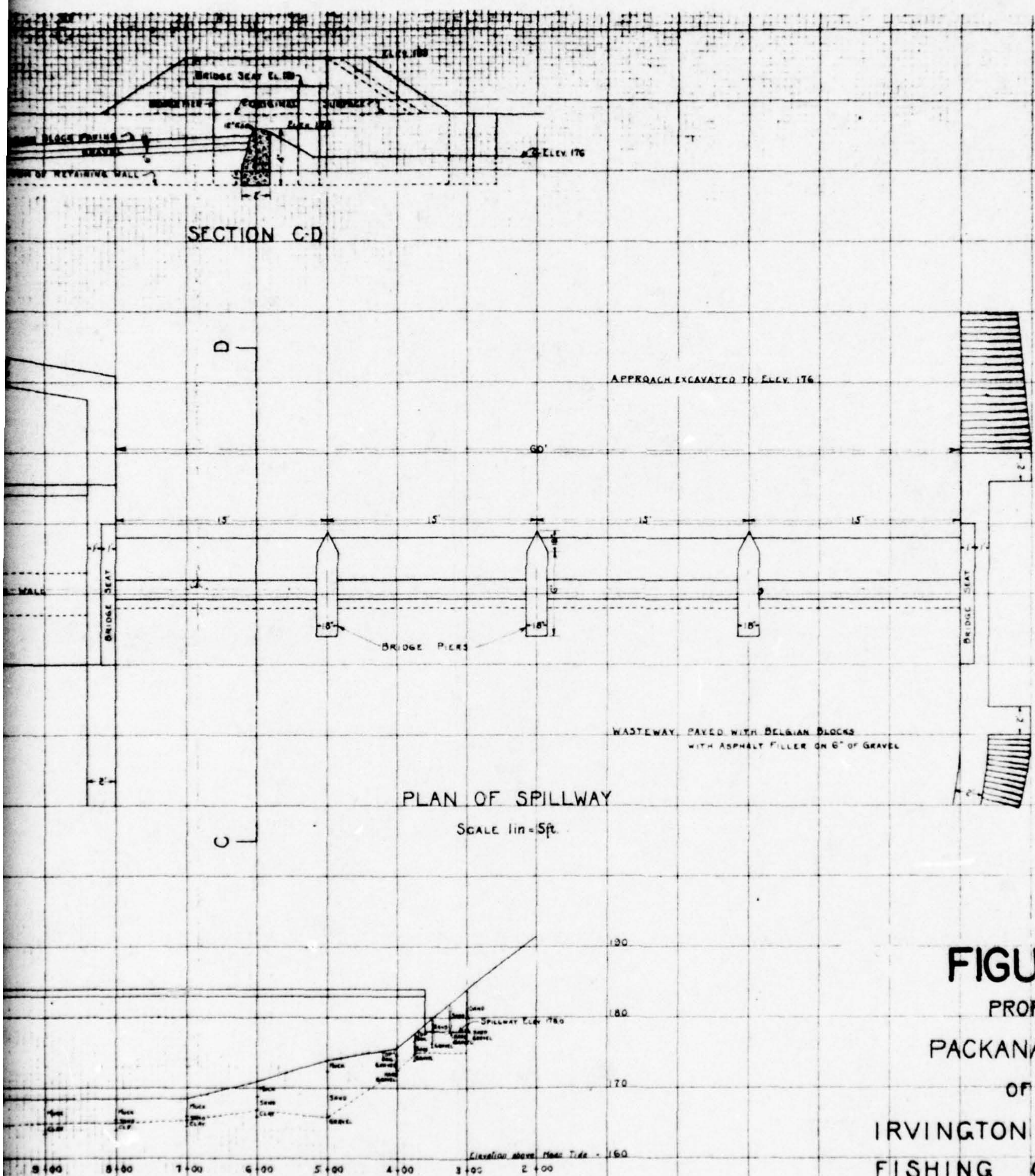


FIGURE 4

PROPOSED

PACKANAC DAM

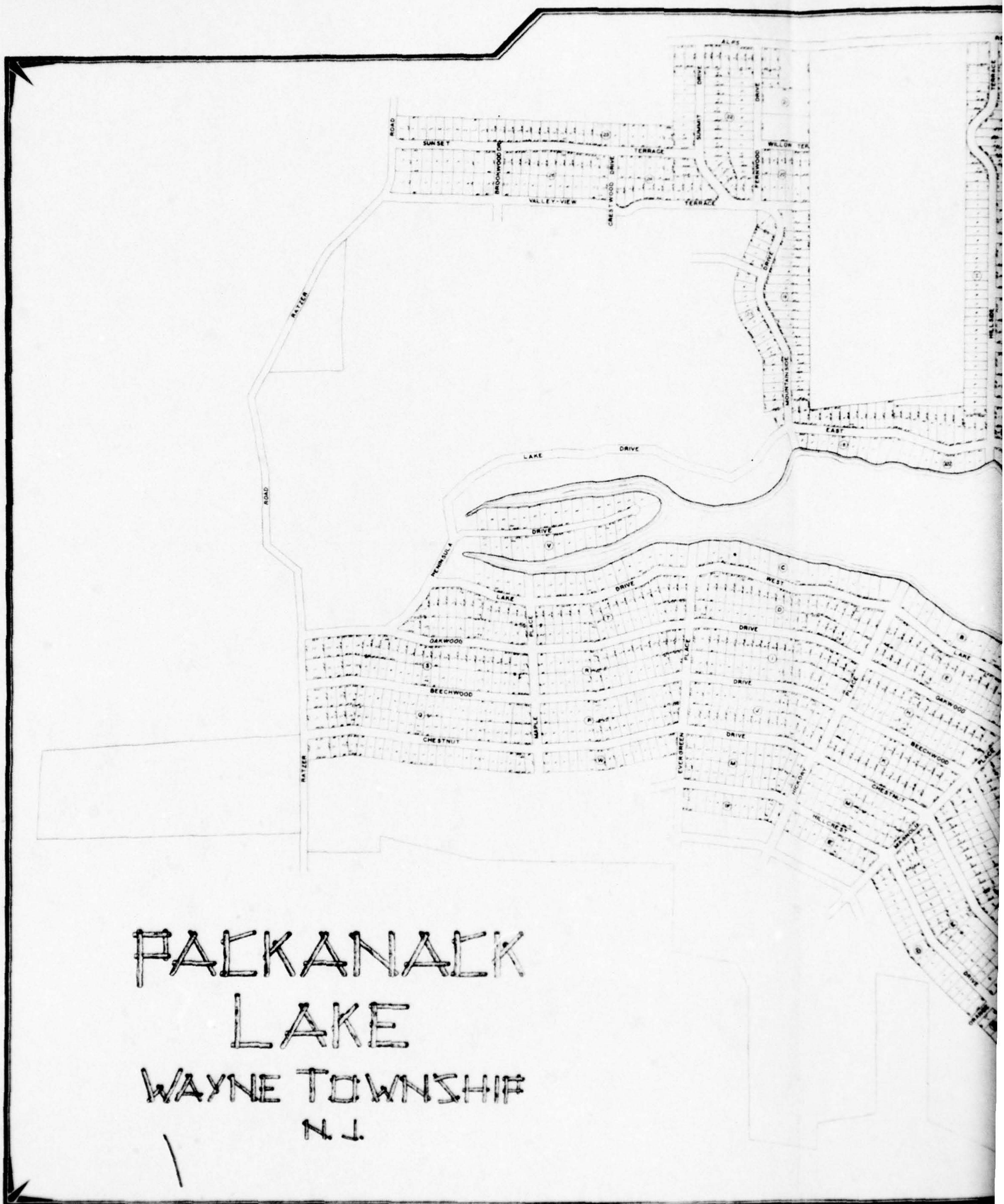
OF THE

IRVINGTON HUNTING & FISHING CORPORATION

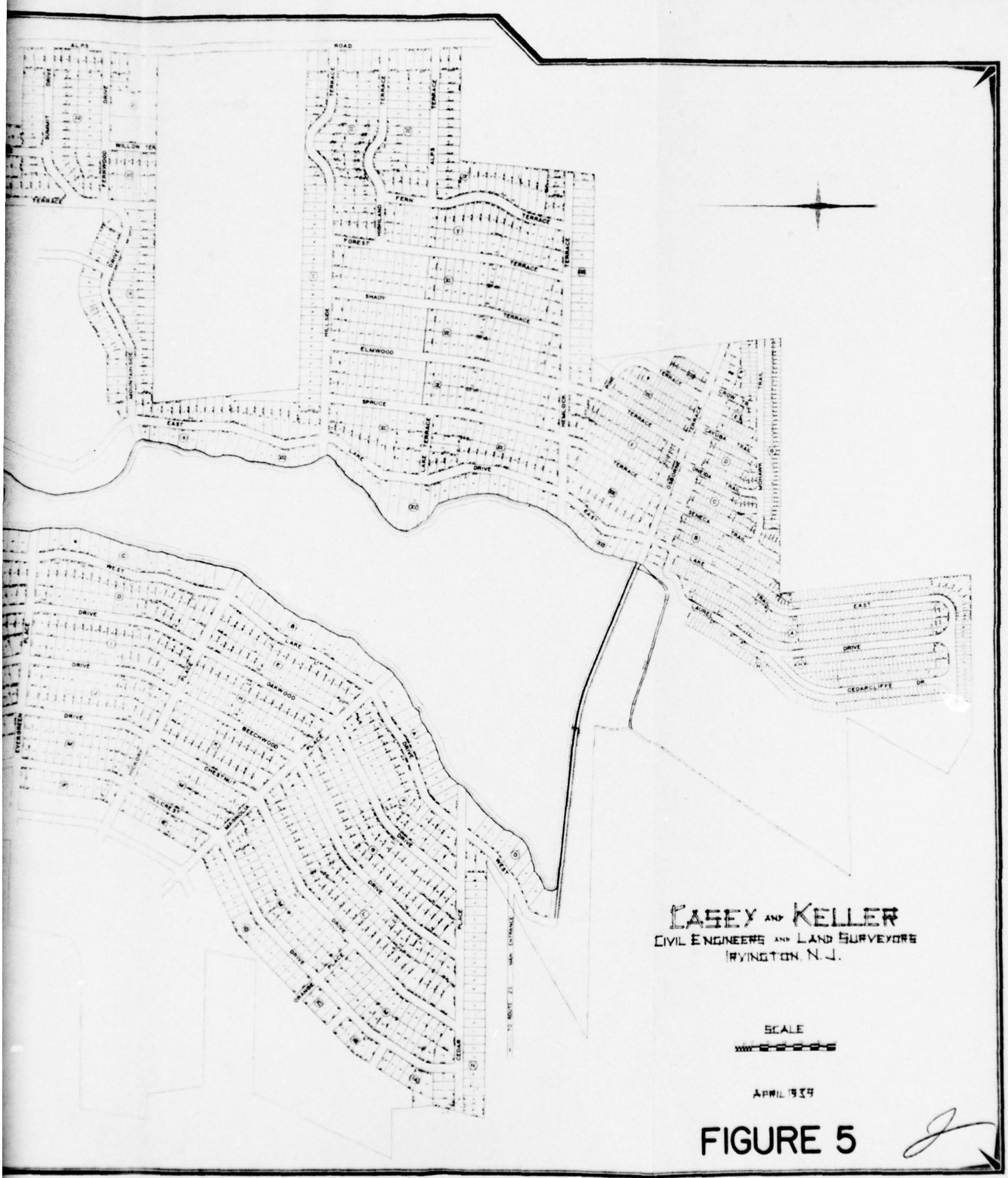
SCALE AS SHOWN APRIL 1, 1926

5

M. R. Sherrerd
Consulting Eng.

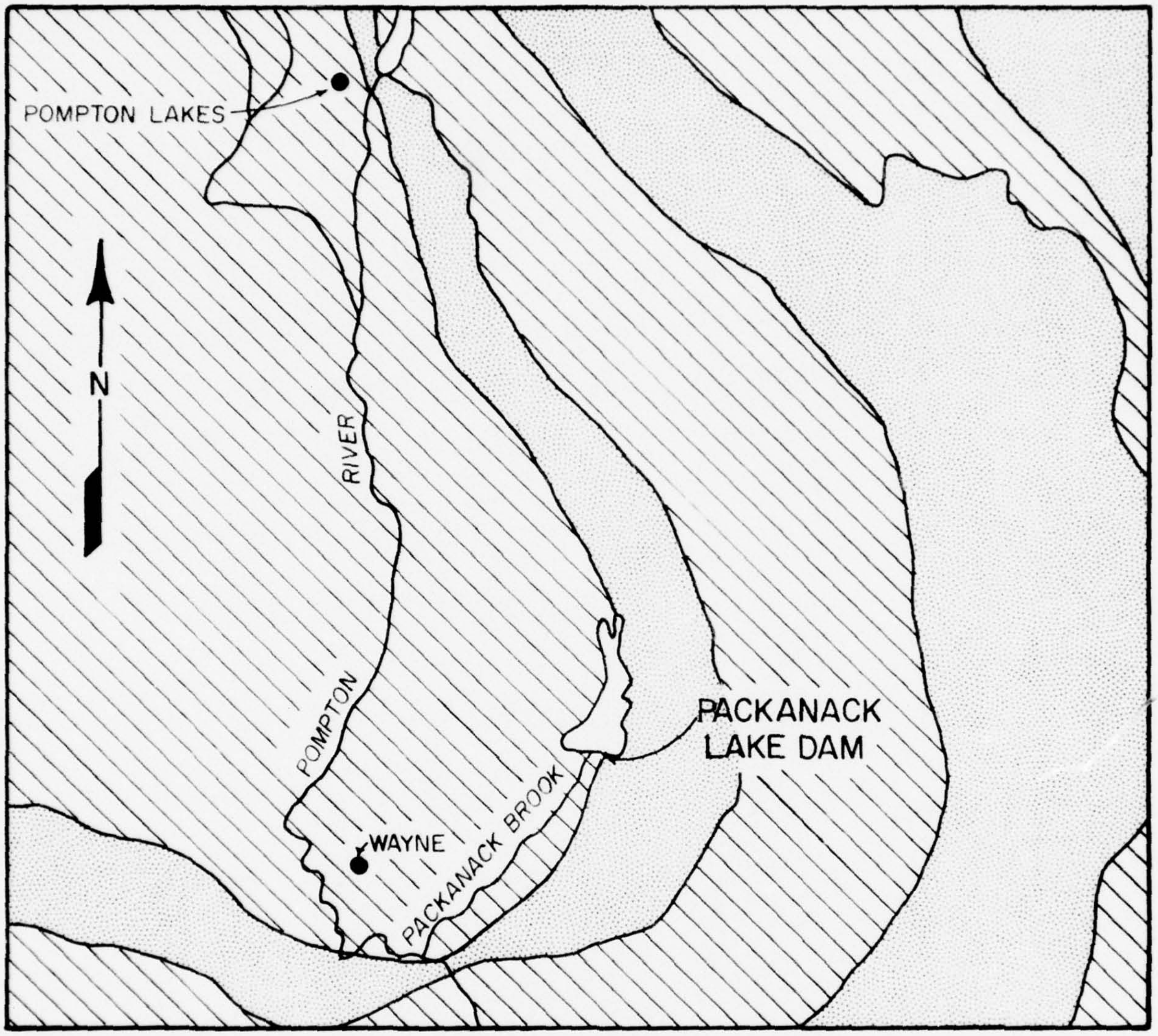


PACKANACK
LAKE
WAYNE TOWNSHIP
N.J.
ALPS
SUNSET
TERRACE
BROOKWOOD DR.
CRESTWOOD DRIVE
VALLEY-VIEW
TERRACE
SUMMIT DRIVE
DRIVE
FERNWOOD
WILLOW TER
ROAD
RATZER
RATZER
ROAD
MOUNTAIN SIDE DRIVE
EAST
LAKE DRIVE
PENINSULA
DRIVE
WEST
LAKE DRIVE
DRIVE
OAKWOOD
BEECHWOOD
CHESTNUT
PLACE
MAPLE
EVERGREEN
PLACE
DRIVE
DRIVE
HICKORY
HILLCREST
CHESTNUT
BEECHWOOD
OAKWOOD
LAKE
MAGNOLIA
HILLSIDE
TERRACE
DRIVE



ALPS ROAD
SUMMIT DRIVE
DRIVE
WILLOW TER
FERNWOOD
TERRACE
DRIVE
MOUNTAINSIDE
EAST
HILLSIDE
TERRACE
TERRACE
ALPS TERRACE
HIGHLAND
FERN TERRACE
FOREST TERRACE
SHADY TERRACE
ELMWOOD
SPRUCE
LAKE TERRACE
LAKE DRIVE
HEMLOCK
TERRACE
OSBORNE
CROW
CAYUGA TRAIL
ONEIDA TRAIL
SENECA TRAIL
MOHAWK
TRAIL
EAST
LAKE
LAUREL
EAST
DRIVE
CEDARCLIFFE DR.
WEST
DRIVE
DRIVE
DRIVE
LAKE
OAKWOOD
BEECHWOOD
CHESTNUT
HICKORY
HILLCREST
EVERGREEN
PLACE
PLACE
PLACE
DRIVE
DRIVE
DRIVE
DRIVE
WEST
ORANGE
PLACE
CEDAR
PLACE
MAIN ENTRANCE
TO ROUTE 23
CASEY AND KELLER
CIVIL ENGINEERS AND LAND SURVEYORS
IRVINGTON, N. J.
SCALE
APRIL 1939

FIGURE 5



SCALE 1 INCH = 1 MILE

LEGEND:

(stippled)	BASALT FLOWS	FINE-GRAINED TRAP ROCK IN EXTENSIVE FLOWS.
(hatched)	BRUNSWICK FORMATION	SOFT RED SHALE WITH SANDSTONE BEDS.

FIGURE 6

GEOLOGIC MAP

APPENDIX

FIELD INSPECTION REPORT

Check List
Visual Inspection
Phase 1

Coordinators: Mr. John Garofalo, Mr. Larry Woscyna, New Jersey DEP

Name Dam: Packanack Lake Dam   County: Passaic   State: New Jersey

Date(s) Inspection: 1/12/78   Weather: Clear   Temperature: 30's

Pool Elevation at Time of Inspection: 178 M.S.L.   Tailwater at Time of Inspection: ______ M.S.L.

Inspection Personnel:

Mr. Jay Williams	Mr. Gurbaksh Sanghera	
Mr. Lee DeHeer		
Mr. Stefan Manea		

Mr. Gurbaksh Sanghera Recorder

Accompanied by:

Mr. John Garofalo, New Jersey Department of Environmental Protection
Mr. Larry Woscyna, New Jersey Department of Environmental Protection
Mr. George Luckman, Manager, Packanack Lake Country Club and Community Association
Mr. William Ragozini, Pandullo-Quirk Associates, Wayne, N.J.

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS	None Noted.	None.
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	None Noted.	None.
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES	Some embankment erosion was noted adjacent to the spillway. Minor sloughing noted at several other locations.	Construction of a wing wall or addition of heavy riprap on the upstream embankment near the spillway would be desirable. Minor sloughing should be repaired as needed.
VERTICAL AND HORIZONTAL ALINEMENT OF THE CREST	No displacement noted.	None.
RIPRAP FAILURES	None Noted.	None.

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM	No problems were noted at the junction of the embankment with the abutments. Additional protection is merited at the junction of the embankment and the spillway.	See comments under "Sloughing or Erosion of Embankment and Abutment slopes.
ANY NOTICEABLE SEEPAGE	None Noted.	None.
STAFF GAGE AND RECORDER	No gages are in use at this site.	None.
DRAINS	No drains noted during inspection or included in plans.	None.

UNGATED SPILLWAY

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE WEIR	A considerable amount of spalling and deterioration has occurred on the spillway. The downstream end has been undermined.	The spillway should be repaired.
APPROACH CHANNEL	None.	None.
DISCHARGE CHANNEL	An 18 inch sanitary sewer pipe crosses the discharge channel just below the concrete weir. This pipe partially blocks the channel and directs flow toward the stream bed, causing excessive erosion. Wing walls below the spillway contract flow from a sixty feet width to a fifteen feet width, causing erosion in the contraction area.	Relocation of the sewer pipe, and removal of the wing wall contraction would increase the discharge capacity and alleviate erosion problems to some extent.
BRIDGE AND PIERS	The spillway structure is bridged. The bridge is supported by three concrete piers. The piers and stoplog supports could cause excessive energy dissipation and reduce discharges.	The stoplog supports should be removed, and the piers should either be streamlined or the bridge replaced by a clear span structure.

OUTLET WORKS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	The outlet conduit is a 20 inch cast iron drain pipe with an 18 inch valve opening. The pipe intake structure is at the bottom of the reservoir and was not inspected.	The drain pipe is operated occasionally to lower the reservoir.
INTAKE STRUCTURE	Same as above.	None.
OUTLET STRUCTURE	The cast iron pipe emerges downstream of the dam and discharges into the original stream bed.	None.
OUTLET CHANNEL	Same as above.	None.
EMERGENCY GATE	None.	None.

RESERVOIR

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	Flat slopes - no signs of sliding	None.
SEDIMENTATION	Sediment is removed periodically from storm sewer outfall areas.	None.

DOWNSTREAM CHANNEL

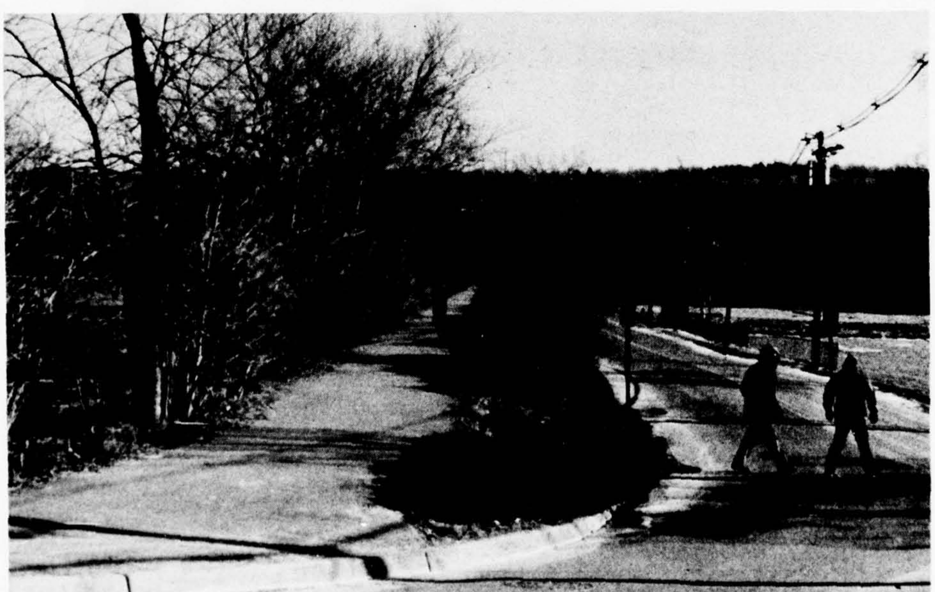
VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION (OBSTRUCTIONS, DEBRIS, ETC.)	The downstream channel is a fifteen feet wide natural watercourse. No problems were noted except for the sanitary sewer & sudden constriction described under spillway section.	None.
SLOPES	No problems were observed.	None.
APPROXIMATE NO. OF HOMES AND POPULATION	The floodplain directly downstream of the dam is wide. Some homes are located within about one-half mile of the dam. Beyond one-half mile are several residential areas with about 50 to 100 homes in the flood plain (Approximately 200 to 500 people).	The Superintendent of Public Works and the Police Captain of Wayne Township monitor flood situations and maintain contact with Civil Defense personnel.

ITEM	REMARKS
MONITORING SYSTEMS	None.
MODIFICATIONS	None.
HIGH POOL RECORDS	No official records for this site. Unofficial information of a reservoir elevation just below the bridge girders. This is a head of 3 feet above the spillway crest, corresponding to a discharge of about 780 cfs.
POST CONSTRUCTION ENGINEERING STUDIES AND REPORTS	Inspections were made in 1969 and in 1976. Copies of these reports are included in the appendix.
PRIOR ACCIDENTS OR FAILURE OF DAM DESCRIPTION REPORTS	In 1971, a water main within the dam ruptured.
MAINTENANCE OPERATION RECORDS	No records available.

ITEM	REMARKS
DESIGN REPORTS	Packanack Lake Dam was completed in October, 1927. Construction reports on the dam consis of photographs and monthly progress which list only percent completion of construction wor A copy of the construction specifications was also included in the available information.
GEOLOGY REPORTS	Shallow subsurface explorations along the proposed axis of the dam were excavated through several feet of muck and terminated in clay.
DESIGN COMPUTATIONS HYDROLOGY & HYDRAULICS DAM STABILITY SEEPAGE STUDIES	½PMF - Inflow peak 5,836; Routed outflow peak - 2,468 cfs (184.1 feet above MSL) This discharge overtops the dam. The dam embankment appears to be stable at normal reservoir levels. No seepage studies were made.
MATERIALS INVESTIGATIONS BORING RECORDS LABORATORY FIELD	None.
POST-CONSTRUCTION SURVEYS OF DAM	See Figure 3.
BORROW SOURCES.	Local.

PHOTOGRAPHS

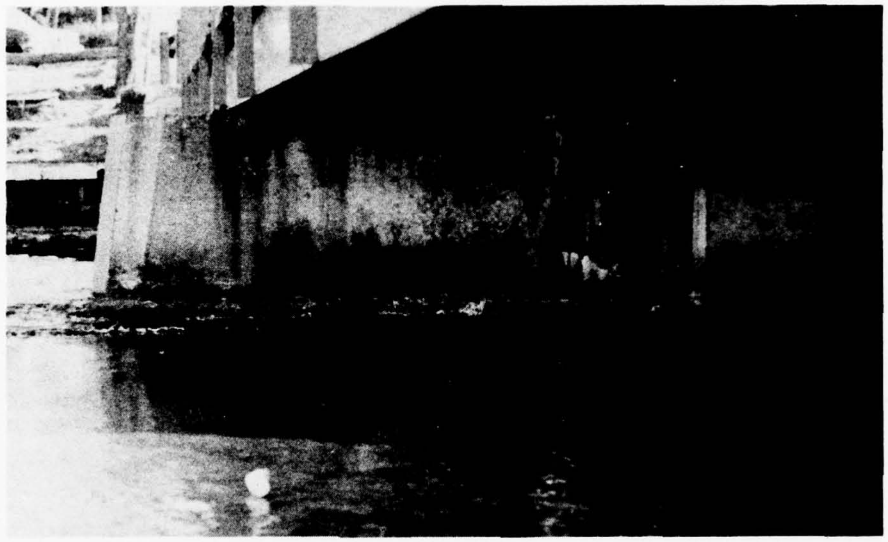
CLOSEUP OF DAM
IMMEDIATELY WEST OF SPILLWAY



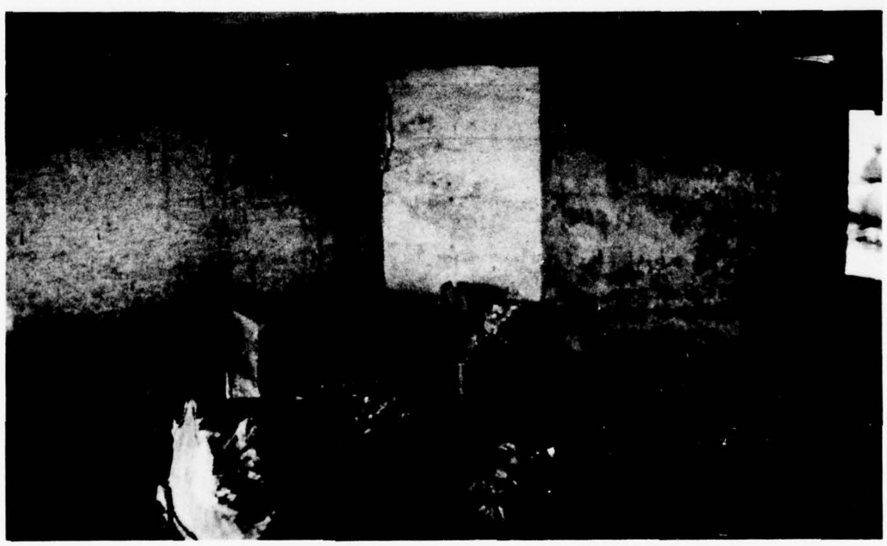
WEST END OF DAM

DOWNSTREAM END OF SPILLWAY LOOKING EAST

DOWNSTREAM END OF SPILLWAY LOOKING WEST



CLOSEUP OF BRIDGE PIER IN SPILLWAY



CLOSEUP OF BRIDGE PIER IN SPILLWAY

CHANNEL DOWNSTREAM OF SPILLWAY

EAST END OF DAM

PREVIOUS INSPECTION REPORTS

MORRIS R. SHERRERD, C. E.
CONSULTING ENGINEER
20 CLINTON STREET
NEWARK, N. J.

April 10th, 1926.

RECEIVED
APR 12 1926
Department of
Conservation & Development

The Department of Conservation and Development,
Division of Waters, State Office Building,
Trenton, N. J.

Gentlemen:

The Irvington Hunting & Fishing Corporation desires to construct a dam on a stream tributary to the Pompton River, near the Village of Wayne, in the Township of Wayne, in the State of New Jersey, to known as Packanac Dam. It submits herewith the following information and respectfully requests the issue of a permit by your Department.

Area of watershed	2.4 square miles.
Maximum depth of pond	10.8 feet.
Area of water surface	82 acres.

Capacity of spillway at 2 feet head 523 cubic feet per second computed by Francis formula.

Capacity of blow-off gate at 12 ~~13~~ feet head 41. ~~4.6~~ cubic feet per second computed by Hazen-Williams formula.

The character of the foundation is clay determined by earth auger borings and washed borings.

It submits the attached papers:

✓ 1. Main requirements of specifications for structure.

✓ 2. Location plan.

✓ 3. Drawings showing

✓ a. Proposed lake.

✓ b. General plan of dam and appurtenances.

✓ c. Longitudinal section and cross sections of dam and spillway.

The drawings and specifications have been prepared by Morris R. Sherrerd, Consulting Engineer, 20 Clinton Street, Newark, N.J.

Very truly yours,

Morris R. Sherrerd
Consulting Engineer for

IRVINGTON HUNTING & FISHING CORPORATION,
998 Springfield Avenue,
Irvington, N. J.

A14

MORRIS R. SHERRERD, C. E.
CONSULTING ENGINEER
20 CLINTON STREET
NEWARK, N. J.

April 10, 1926.

Memorandum regarding specifications
to accompany application of the
Irvington Hunting & Fishing Corp.,
for the approval of the construction of
a dam at Wayne, N. J.

The specifications for this work will definitely provide for drainage ditches on each side of the location of the dam, for daming the main brook below the point at which the dam will cross it, constructing pumping sump above this dam and pumping the water from the brook above the dam over the dam in an effort to entirely drain the location of the dam itself and the swamp above.

The excavation of the leaf mold and sand overlying the clay at the dam site with the possible exception of leaving part of this material on the down stream side which might be suitable for the construction of the down stream side of the embankment.

The bringing in of impervious material and placing the same in the embankment in layers not exceeding six inches and thoroughly compacting the same by rolling.

The construction of carefully spaded clay core for the dam to be made from suitable clay and gravel material which are near at hand.

The careful construction of blow off pipe through the dam at a point a short distance easterly of the main stream. This pipe to be carried on concrete piers and the space between the concrete piers and surrounding the pipe for at least a foot above the same to be filled with clay puddle and incorporated with puddle cut off.

The concrete for the overflow to be made of Portland Cement of a mix 1:2:3 and the paving between wing walls and down stream side of the overflow channel to be carefull laid and grouted with asphalt.

The entire dam to be traversed by a roadway made of gravel thoroughly compacted.

The upstream slope of the dam to be rip-rap and the down stream side to be seeded. Slopes of embankment to be at least 1:2 and a portion of the excavated material will be used on the down stream embankment to flatten that part of the slope.

Detail copies of the specifications will be filed with the Department of Conservation and Development before the contract for the work is awarded provided plans for construction are approved by the Department.

Respectfully submitted,

Morris R Sherrerd

Consulting Engineer.

DEPARTMENT OF ENVIRONMENTAL PROTECTION
DIVISION OF WATER RESOURCES
BUREAU OF WATER CONTROL
P.O. BOX 2809
TRENTON, NEW JERSEY 08625

RECEIVE[illegible]
JAN 12 8 2[illegible]
DEPT. ENV'R PRO[illegible]
DIV. OF WATER RESO[illegible]

CONDITION REPORT - DAMS

Dam Application No.__________ Date of Inspection December 2, 1975

Name of Dam Packanack Lake Dam

Owner's Name Packanack Lake County Club & Community Assn.

Address P. O. Box 669, Packanack Lake

Wayne, New Jersey 07470

Comment on the following items in accordance with the instructions enclosed:

A. Earthfill and/or Timber Dams

1. Maintenance - Good. Shrubs and trees are trimmed and cut annually. There are a few large trees whose roots are penetrating the dam, riprap, and asphaltic paved footpath on top of dam.
2. Condition - Good. Embankment is in good condition. No sign of erosion or seepage. Roots from the larger trees are causing minor shifting of riprap and cracks and deterioration of asphaltic pavement.

3. Other - Spillway - see paragraph "C".

B. Masonry and Concrete Dams

1. (Not Applicable)

2.

3.

4.

5.

6.

C. Channels, Stilling Basins and Surrounding Areas

1. Spillway - Some minor concrete spalling and erosion at water line. Some slight undercutting of concrete apron lip downstream side of spillway - does not appear significant at this time but should be watched.

2. Channels - Inlet channels show earth erosion. Some minor evidence of erosion on outlet channel also but it does not represent a hazard to dam or spillway.

3. Stilling Basin - The two inlet fingers of the lake function as sedimentation basins and are dredged annually.

D. Mechanical Equipment

1. An 18-inch valve and pipe drain are used occasionally to lower lake level, when authorized, below spillway level.

2.

E. Miscellaneous

1. Sidewalk bridge over Spillway Steel girders are showing severe corrosion, If bridge failed it would block spillway flow.

2. Osborne Terrace Bridge - Roadway bridge over spillway carries medium to heavy traffic as a through street. Concrete pier supports are deteriorating, spalling and eroding in a few places at girder bearing pads.

Girders are sound but corrosion is occurring particularly on bottom flanges and at bearings.

Stringers are of steel and wood and are badly deteriorated. Rust and wood rot are in advanced stages. Wood decking is rotting and in poor condition. Asphaltic macadam pavement over wood decking shows longitudinal and lateral cracks and one hole has been recently patched.

Failure of bridge system will likely gradually begin to occur now. This bridge has had no major repairs since Passaic County worked on it in early 1950's.

CONCLUSION

A. I certify that the above dam was personally inspected by me and was found to be in (good, fair, poor) condition. (Circle o

B. I recommend that the following repairs be made immediately.

1. Both the sidewalk bridge and the main roadway bridge carrying Osborne Terrace over the spillway are in need of heavy maintenance or reconstruction. Reinforced concrete stringers and deck should be considered because of the location above spillway.
2.
3.

C. The following improvements should also be undertaken.

1. Large diameter trees on dam should be systematically removed.
2.
3.

Inspected by: Ernest Chrisbacher

Consulting Engineer: Pandullo Qui Associates

Address: 40 Galesi Drive

Wayne, New Jersey 07470

SEAL

Telephone: 201-785-2410

N.J.P.E. License No. 11661

Date: December 2, 1975

d. Photographs of the upstream and downstream faces of the embankment, main spillway and emergency spillway noting date taken.

Use additional sheets when necessary.

Inspected by George L. Sullivan

Consulting Engineer

N.J. License No. 12777

Date: July 14 1969

Annual Report - Dams

Application No. 75 For Year: 19 69

Name of Dam PACKANACK LAKE DAM Date of Inspection: 7/14/69

Owner, Name Packanack Lake C.C. & Comm. Assn.

Address Wayne, N.J.

AUG 5 '69

Description of condition of the following:

1. Embankment (Erosion, seepage, etc.)
   Embankment in excellent condition. No sign of erosion or seepage.

2. Spillway (Concrete spalling, timber rotting, leakage, etc.)
   Spillway is concrete. No sign of spalling or seepage.

3. Emergency Spillway (Erosion, growth of sod, riprap, etc.)
   Emergency spillway consists of a 18 inch valve which is in good condition and is operated from time to time to drop lake level below spillway.

4. Outlet Works (Operational condition of valves or grates, condition of pipe, etc.)
   Same as Item 3.

5. Inlet streams (Silt deposition, etc.)
   Silt is periodically removed from inlet stream and many storm water drains.

6. Outlet stream (Scouring, undercutting of dam, condition of stilling basin, etc.)
   Outlet stream is also cleaned from time to time. There is no sign of undercutting of the dam. There is no stilling basin.

7. General
   a. Did flood waters overtop dam during period of report? If so, at what stage and date thereof.
      No flood waters ever went over the top of the dam.

   b. Report on any other condition not covered above.
      A 10 foot macadam foot path covers top of the dam and is kept in good repair.

   c. In your opinion, does existing condition warrant repairs? If so, where and to what extent.
      Repairs are made as needed. Nothing to recommend at this tim

HYDROLOGIC AND HYDRAULIC CALCULATIONS

JUSTIN & COURTNEY
DIVISION OF O'BRIEN & GERE ENGINEERS

SUBJECT	SHEET	BY	DATE	JOB NO
PACKANACK LAKE DAM	1	DBC	1/25/78	1800.001.19

PROBABLE MAXIMUM FLOOD CALCULATION

DRAINAGE AREA = 1.88 SQ. MI.

FROM HYDROMETEOROLOGICAL REPORT #33,

6 HOUR - 10 SQ MI PMP = 26"
ZONE #6

SINCE THE DRAINAGE AREA IS UNDER 10 SQ. MI., NO REDUCTION REFLECTING BASIN SIZE IS INCORPORATED.

	FACTOR		
6 HR PMP	1.00	× 26"	= 26.0"
12 HR PMP	1.09	× 26"	= 28.3"
24 HR PMP	1.17	× 26"	= 30.4"
48 HR PMP	1.26	× 26"	= 32.8"

A REDUCTION OF 20% IS INCLUDED TO ACCOUNT FOR PROBABLE MISALIGNMENT OF BASIN & STORM ISOHYETALS.

6 HR PMP	- 20.8"			
12 HR PMP	- 22.6"	(22.6-20.8)/6 = 1.8/6	= .3 IN/HR*	
24 HR PMP	- 24.3"	(24.3-22.6)/12 = 1.7/12	= .14 IN/HR*	
48 HR PMP	- 26.2"	(26.2-24.3)/24 = 1.9/24	= .08 IN/HR*	

*THESE ARE NEGLECTED

JUSTIN & COURTNEY
DIVISION OF O'BRIEN & GERE ENGINEERS

SUBJECT	SHEET	BY	DATE	JOB NO.
PACKANACK LAKE DAM	2	DBC	1/26/78	1800 001.

'PMP' DETERMINATION FOR RAINFALL INCREMENTS (MAX. 6 HRS.)

TIME (HRS)	Σ % 6HR 'PMP'	Σ 6HR 'PMP'	INCREMENTAL 'PMP'
.3	22%	4.6	4.6
.6	34%	7.1	2.5
.9	47%	9.8	2.7
1.2	53%	11.0	1.2
1.5	58%	12.1	1.1
1.8	62%	12.9	0.8
2.1	66%	13.7	0.8
2.4	69%	14.4	0.7
2.7	72%	15.0	0.6
3.0	75%	15.6	0.6
3.3	78%	16.2	0.6
3.6	81%	16.8	0.6
3.9	84%	17.5	0.7
4.2	86%	17.9	0.4
4.5	88%	18.3	0.4
4.8	91%	18.9	0.6
5.1	93%	19.3	0.4
5.4	96%	20.0	0.7
5.7	98%	20.4	0.4
6.0	100%	20.8	0.4

NOTE: .3 HOUR RAINFALL INCREMENTS WERE USED, DUE TO THE RAPID Tc.

SUBJECT	SHEET	BY	DATE	JOB NO
PACKANACK LAKE DAM	3	DBC	1/26/78	1800.001.19

ADJUSTED 6 HR SEQUENCE (THIRD QUARTILE)

TIME (HRS)	INCREMENTAL 'PMP'	Σ 6HR 'PMP'
.3	.4	.4
.6	.4	.8
.9	.4	1.2
1.2	.4	1.6
1.5	.4	2.0
1.8	.7	2.7
2.1	.7	3.4
2.4	.7	4.1
2.7	.8	4.9
3.0	.8	5.7
3.3	1.1	6.8
3.6	1.2	8.0
3.9	2.5	10.5
4.2	4.6	15.1
4.5	2.7	17.8
4.8	.6	18.4
5.1	.6	19.0
5.4	.6	19.6
5.7	.6	20.2
6.0	.6	20.8

JUSTIN & COURTNEY
DIVISION OF O'BRIEN & GERE ENGINEERS

SUBJECT	SHEET	BY	DATE	JOB NO.
PACKANACK LAKE DAM	4	DEC	1/26/78	13000.001.1

DRAINAGE BASIN SOIL GROUP 'C' AMC II

LAND USE: 75% HOUSING (MOSTLY SFD) CN85

25% WOODS CN73

AVERAGE CN = .75 × 85 + .25 × 73 = 82

TIME (HR)	PMF RAINFALL		DIRECT RUNOFF		LOSSES	
	INCR	Σ	INCR.	Σ	INCR.	Σ
.3	.4	.4	.00	0.0	.4	.4
.6	.4	.8	.05	.05	.35	.75
.9	.4	1.2	.15	.20	.25	1.00
1.2	.4	1.6	.20	.40	.20	1.20
1.5	.4	2.0	.25	.65	.15	1.35
1.8	.7	2.7	.50	1.15	.20	1.55
2.1	.7	3.4	.55	1.70	.15	1.70
2.4	.7	4.1	.60	2.30	.10	1.80
2.7	.8	4.9	.70	3.00	.10	1.90
3.0	.8	5.7	.70	3.70	.10	2.00
3.3	1.1	6.8	1.05	4.75	.05	2.05
3.6	1.2	8.0	1.10	5.85	.10	2.15
3.9	2.5	10.5	2.40	8.25	.10	2.25
4.2	4.6	15.1	4.50	12.75	.10	2.35
4.5	2.7	17.8	2.65	15.40	.05	2.40
4.8	.6	18.4	.55	15.95	.05	2.45
5.1	.6	19.0	.55	16.50	.05	2.50
5.4	.6	19.6	.55	17.05	.05	2.55
5.7	.6	20.2	.55	17.60	.05	2.60
6.0	.6	20.8	.55	18.15	.05	2.65

SUBJECT	SHEET	BY	DATE	JOB NO.
PACKANACK LAKE DAM	5	DBC	1/25/78	1800.001.1

ESTIMATION OF "T_c"

BUREAU OF PUBLIC ROADS

$$T_c = \left(\frac{11.9 L^3}{H}\right)^{.385} \qquad L \approx 1.36 \text{ MILE} \qquad H = 270 - 178' = 92'$$

$$T_c = \left(\frac{11.9 \times 1.36^3}{92}\right)^{.385} = .65 \text{ HRS.}$$

SCS UPLAND METHOD

average land slope ≈ 4.0%
velocity = 3.6 fps (paved areas and grassed waterw... average)

t_c 7200/3.6×3600 : .56 HRS.

SCS CURVE NUMBER METHOD

$$L = \frac{\ell^{0.8}(S+1)^{0.7}}{1900\, Y^{0.5}} = \text{Lag}$$

$\ell = 7200'$ $S = \frac{1000}{CN} - 10 = \frac{1000}{82} - 10 = 2.2$ (CN=82)

$Y = 4.0\%$

$$L = \frac{7200^{.8} \times 3.2^{.7}}{1900 \times 4.0^{.5}} = .72 \text{ HRS}$$

$$T_c = L/.6 = 1.21 \text{ HRS.}$$

SUBJECT	SHEET	BY	DATE	JOB NO
PACKANACK LAKE DAM	6	DBC	1/25/78	1800.001.

SUMMARY OF T_c VALUES

BPR — .65 HRS. OK

SCS (UPLAND) — .56 HRS. OK   USE T_c = .60 HRS

SCS (CN METHOD) — 1.21 HRS. (TOO HIGH)

TRIANGULAR UNIT HYDROGRAPH

$T_p = \frac{D}{2} + .6 T_c$ $(D = .5)$

$T_p = .25 + .6 \times .6 = .61$ HRS. USE .6 HRS.

$T_b = 2.67 \times T_p = 1.6$ HRS.

$q_p = 484\ A / T_p = 484 \times 1.88 / .6 = 1517$ CFS

DUE TO THE RAPID TIME OF CONCENTRATION, A tr OF .3 HOURS IS USED TO MODEL THE RUNOFF.

JUSTIN & COURTNEY, INC.
Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET No. 7 OF ____

DATE 1/25/78

COMP. BY DBC

CHECKED BY ____

NAME OF CLIENT Corps of Engrs.

PROJECT Packanack Lake Dam (Unitgraph)

LW

$T_P = .6$ HRS. $q_P = 1517.$ CFS

T/Tp	q/qp	TIME (HRS)	DISCHARGE (CFS)	ADJUSTED q's FOR EXACTLY 1" OF RUNOFF
0.0	0.00	0.00	0	0
0.1	0.015	0.06	23	23
0.2	0.075	0.12	114	112
0.3	0.16	0.18	243	239
0.4	0.28	0.24	425	418
0.5	0.43	0.30	652	641
0.6	0.60	0.36	910	895
0.7	0.77	0.42	1168	1149
0.8	0.89	0.48	1350	1328
0.9	0.97	0.54	1471	1447
1.0	1.00	0.60	1517	1492
1.1	0.98	0.66	1487	1463
1.2	0.92	0.72	1396	1373
1.3	0.84	0.78	1274	1253
1.4	0.75	0.84	1138	1120
1.5	0.66	0.90	1001	985
1.6	0.56	0.96	850	836
1.8	0.42	1.08	637	627
2.0	0.32	1.20	485	477
2.2	0.24	1.32	364	358
2.4	0.18	1.44	273	269
2.6	0.13	1.56	197	194
2.8	0.098	1.68	149	147
3.0	0.075	1.80	114	112
3.5	0.036	2.10	55	54
4.0	0.018	2.40	27	27
4.5	0.009	2.70	14	14
5.0	0.004	3.00	6	6
5.5	0.00	3.30	0	0

1" DBC

$\sum_{T=0.0}^{.96} q = 14594$ $\sum_{T=.96}^{1.80} q = 2587$ $\sum_{T=1.80}^{3.30} q\ 159$

DIVIDED DISCHARGE VALUES BY 1.016

INCHES OF RUNOFF = $\Sigma q \times \Delta T / 645.6 \times D.A.$

RUNOFF = $(14594 \times .06 + 2587 \times .12 + 159 \times .3)/(645.6 \times 1.88) = \underline{1.016"}$

DIVISION OF O'BRIEN & GERE ENGINEERS

SUBJECT	SHEET	BY	DATE	JOB NO
PACKANACK LAKE DAM	8	DBC	1/26/78	1800.001.

TIME ENDING	INCREMENTAL	q_p FOR	INCREMENTAL HYDROGRAPHS		
HOUR	RUNOFF	INCREMENTS	BEGIN	PEAK	END
~~.3~~	~~0.00~~	~~0~~	—	—	—
.6	0.05	76	0	.6	3.3
.9	0.15	228	.3	.9	3.6
1.2	0.20	303	.6	1.2	3.9
1.5	0.25	379	.9	1.5	4.2
1.8	0.50	759	1.2	1.8	4.5
2.1	0.55	834	1.5	2.1	4.8
2.4	0.60	910	1.8	2.4	5.1
2.7	0.70	1062	2.1	2.7	5.4
3.0	0.70	1062	2.4	3.0	5.7
3.3	1.05	1593	2.7	3.3	6.0
3.6	1.10	1669	3.0	3.6	6.3
3.9	2.40	3641	3.3	3.9	6.6
4.2	4.50	6827	3.6	4.2	6.9
4.5	2.65	4020	3.9	4.5	7.2
4.8	.55	834	4.2	4.8	7.5
5.1	.55	834	4.5	5.1	7.8
5.4	.55	834	4.8	5.4	8.1
5.7	.55	834	5.1	5.7	8.4
6.0	.55	834	5.4	6.0	8.

1800.001.191

JUSTIN & COURTNEY, INC.
Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET NO. 9 OF ___ LW

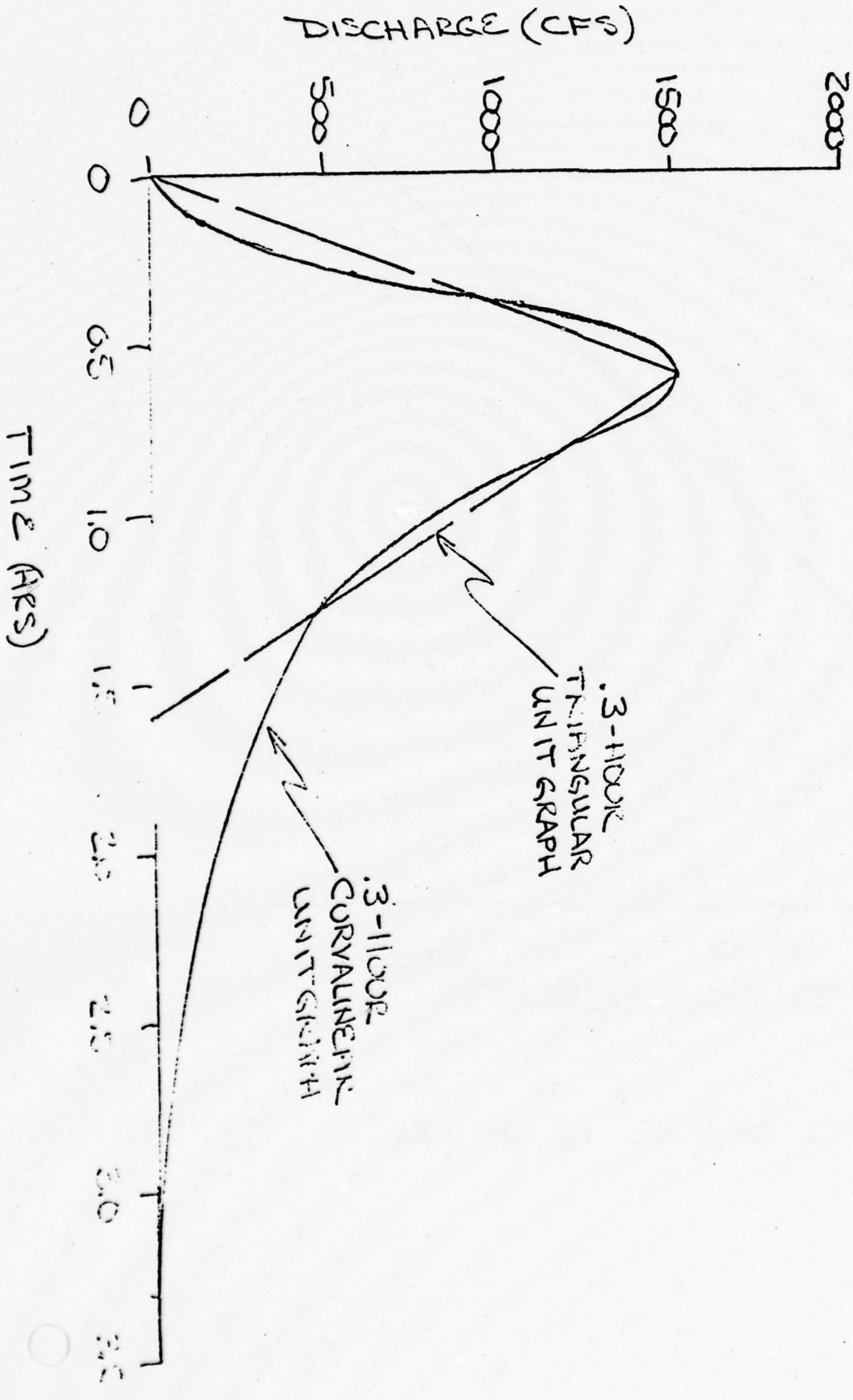
DATE 1/25/78

NAME OF CLIENT: CORPS OF ENGRS.

COMP. BY: DC

PROJECT: PACKANACK LAKE DAM

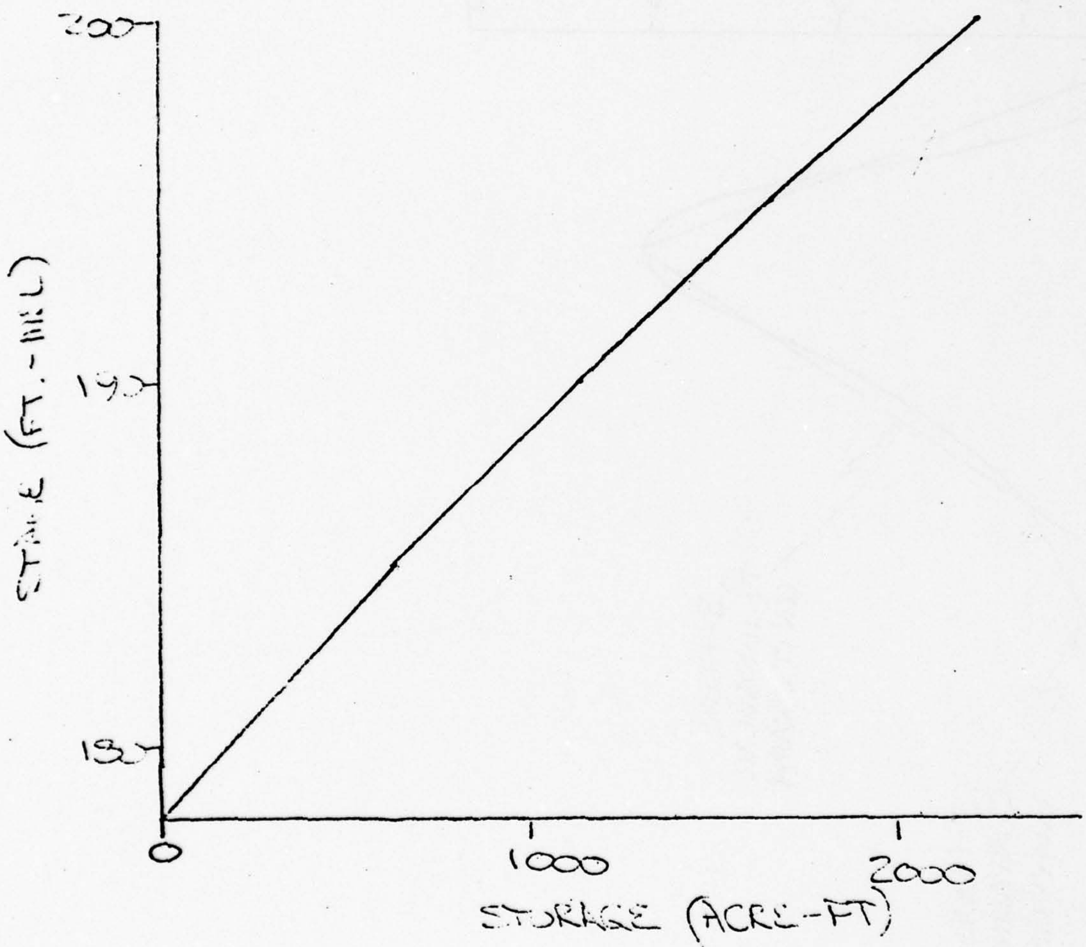
CHECKED BY: LW



JUSTIN & COURTNEY, INC.
Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET NO. 10 OF ____
DATE 1/26/78
COMP. BY DEC
CHECKED BY CW

NAME OF CLIENT Corps of Engrs.
PROJECT Hackanack Lake Dam



STAGE - STORAGE RELATION

JUSTIN & COURTNEY, INC.
Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET NO. 11 OF ____

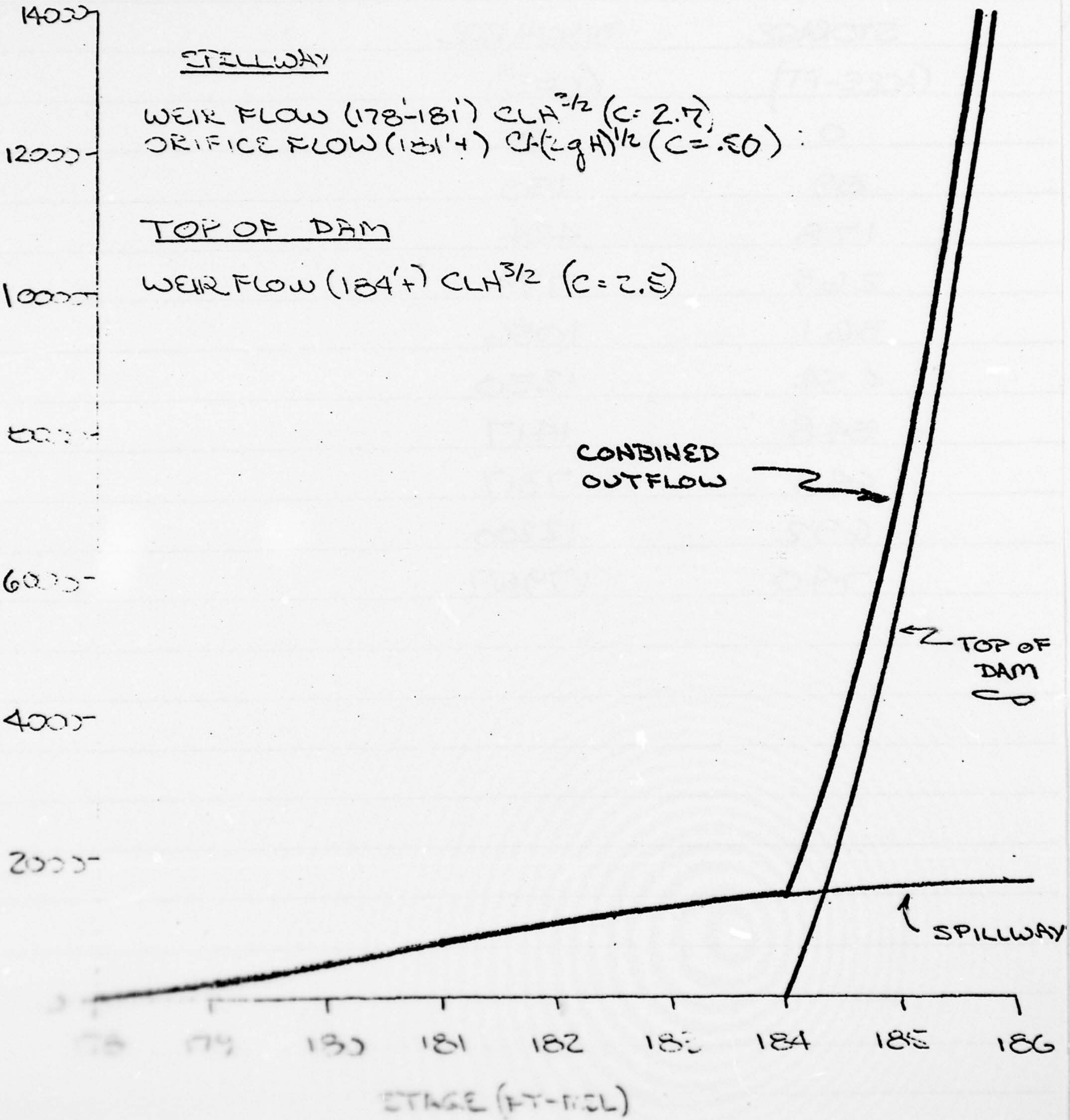
DATE 1/27/78

NAME OF CLIENT: CORPS OF ENGRS

COMP. BY: DJC

PROJECT: PACKANACK LAKE DAM

CHECKED BY: LW



JUSTIN & COURTNEY
DIVISION OF O'BRIEN & GERE ENGINEERS

SUBJECT	SHEET	BY	DATE	JOB NO
PACKANACK LAKE DAM	12	TEC	1/31/79	522.001.

STORAGE (ACRE-FT)	DISCHARGE (CFS)
0	0
89	160
178	424
269	779
361	1056
454	1250
548	1417
644	7317
692	12200
740	17967

DISCHARGE FOR 20 C.I. PIPE

ASSUMED OUTLET CONTROL & CULVERT FLOWING FULL

$$H = \frac{V^2}{2g} + K_e\frac{V^2}{2g} + K_v\frac{V^2}{2g} + h_f$$

H = UPSTREAM HEAD

Ke = ENTRANCE LOSS COEFFICIENT

Kv = VALVE LOSS COEFFICIENT

$$h_f = \frac{29\, n^2 L}{\left(\frac{D}{4}\right)^{1.33}} V^2/2g \qquad \& \quad V^2/2g = Q^2/2gA^2$$

$$H = \left[1 + K_e + K_v + \frac{29\, n^2 L}{(D/4)^{1.33}}\right]\frac{V^2}{2g}$$

$$H = \left[1 + .1 + .1 + \frac{29 \times (.008)^2 \times 75}{(.417)^{1.33}}\right]\frac{Q^2}{2g\frac{D^2}{4}\pi}$$

$$H = .012Q^2 \qquad Q = (85.66H)^{1/2}$$

H feet	9	7	5	3	1	
Q cfs	27.8	24.5	20.7	16.0	9.3	

AT LEVELS CAUSING OVERTOPPING, Q THROUGH THE PIPE IS 25 CFS, INSIGNIFICANT FOR FLOOD FLOWS.

1800.001.191

JUSTIN & COURTNEY, INC
Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET NO. ______ OF ______
DATE 2/12/78
NAME OF CLIENT U.S. CORPS OF ENGRS.
COMP. BY DPC
PROJECT PACKANACK LAKE DAM
CHECKED BY LW

RESERVOIR DRAWDOWN CALCULATIONS

ASSUMPTIONS:

LINEAR VARIATION IN SURFACE AREA FROM 84 ACRES AT THE SPILLWAY CREST TO ZERO ACRES TEN FEET LOWER

ΔH (feet)	Have (feet)	Qave (cfs)	Aaverage (acres)	Δt (hours)	Δt (days)
2	9	27.8	75.6	65.8	2.74
2	7	24.5	58.8	58.1	2.42
2	5	20.7	42.0	49.1	2.05
2	3	16.0	25.2	38.1	1.59
2	1	9.3	8.4	21.9	0.91
				Σ=233	Σ=9.71

$24\overline{)233}$ = 9.7 days ✓

COMPUTER PRINTOUT

FOR THE

PROBABLE MAXIMUM FLOOD

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***  ***********************
HEC-1 VERSION DATED JAN 1973
UPDATED AUG 74
CHANGE NO. 01
****************************

                              ROUTING OF PMF FLOOD
                            PACKANACK LAKE DAM
                       O-BRIEN + GERE--JUSTIN + COURTNEY DIV

                                JOB SPECIFICATION
                  NQ    NHR  NMIN  IDAY   IHR  IMIN METRC  IPLT  IPRT NSTAN
                 100      0     9     1     0     0     0     2     2     0
                                   JOPER     NWT
                                       3       0

          **********        **********        **********        **********        **********

                             SUB-AREA RUNOFF COMPUTATION

                PMF INFLOW HYDROGRAPH TO PACKANACK LAKE
                       ISTAO   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME
                           1       0       0       0       0       0       1

                                  HYDROGRAPH DATA
            IHYDG    IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
                0      -1    1.83    0.00    0.00    0.00   0.000       0       0       0

                                    PRECIP DATA
                             NP   STORM     DAJ     DAK
                             37    0.00    0.00    0.00
                                  PRECIP PATTERN
       .05      0.00       .15      0.00       .20      0.00       .26      0.00       .50      0.00
       .55      0.00       .60      0.00       .70      0.00       .70      0.00      1.05      0.00
      1.10      0.00      2.40      0.00      4.50      0.00      2.65      0.00       .55      0.00
       .55      0.00       .55      0.00       .55      0.00       .55

                                     LOSS DATA
           STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
            0.00    0.00    1.00    0.00    0.00    1.00    0.00    0.00    0.00    0.00

                             GIVEN UNIT GRAPH, NUHGQ=  23
        0.      170.      641.     1225.     1432.     1300.      985.      770.      477.      328.
      232.      156.      112.       81.       54.       38.       27.       20.       14.       10.
        6.        3.        0.
                  UNIT GRAPH TOTALS   8141. CFS OR 1.01 INCHES OVER THE AREA

                                  RECESSION DATA
                      STRTQ=    0.00   QRCSN=    0.00   RTIOR= 1.00

                                END-OF-PERIOD FLOW
                             TIME    RAIN    EXCS    COMP Q
                          1  0  9     .05     .05        0.
                          1  0 18    0.00    0.00        9.
                          1  0 27     .15     .15       32.
                          1  0 36    0.00    0.00       87.
                          1  0 45     .20     .20      171.
```

1	1	30	0.00	0.00	795.
1	1	39	.55	.55	989.
1	1	48	0.00	0.00	1255.
1	1	57	.60	.60	1490.
1	2	6	0.00	0.00	1719.
1	2	15	.70	.70	1888.
1	2	24	0.00	0.00	2085.
1	2	33	.70	.70	2216.
1	2	42	0.00	0.00	2407.
1	2	51	1.05	1.05	2507.
1	2	60	0.00	0.00	2699.
1	3	9	1.10	1.10	2899.
1	3	18	0.00	0.00	3261.
1	3	27	2.40	2.40	3534.
1	3	36	0.00	0.00	4054.
1	3	45	4.50	4.50	4825.
1	3	54	0.00	0.00	6365.
1	4	3	2.65	2.65	8346.
1	4	12	0.00	0.00	10480.
1	4	21	.55	.55	11688.
1	4	30	0.00	0.00	11665.
1	4	39	.55	.55	10325.
1	4	48	0.00	0.00	8769.
1	4	57	.55	.55	6699.
1	5	6	0.00	0.00	5525.
1	5	15	.55	.55	4394.
1	5	24	0.00	0.00	3743.
1	5	33	.55	.55	3274.
1	5	42	0.00	0.00	2990.
1	5	51	0.00	0.00	2731.
1	5	60	0.00	0.00	2526.
1	6	9	0.00	0.00	2119.
1	6	18	0.00	0.00	1667.
1	6	27	0.00	0.00	1172.
1	6	36	0.00	0.00	860.
1	6	45	0.00	0.00	560.
1	6	54	0.00	0.00	383.
1	7	3	0.00	0.00	257.
1	7	12	0.00	0.00	176.
1	7	21	0.00	0.00	117.
1	7	30	0.00	0.00	84.
1	7	39	0.00	0.00	56.
1	7	48	0.00	0.00	39.
1	7	57	0.00	0.00	26.
1	8	6	0.00	0.00	18.
1	8	15	0.00	0.00	11.
1	8	24	0.00	0.00	7.
1	8	33	0.00	0.00	3.
1	8	42	0.00	0.00	2.
1	8	51	0.00	0.00	0.
1	8	60	0.00	0.00	0.
1	9	9	0.00	0.00	0.
1	9	18	0.00	0.00	0.
1	9	27	0.00	0.00	0.
1	9	36	0.00	0.00	0.
1	9	45	0.00	0.00	0.
1	9	54	0.00	0.00	0.
1	10	3	0.00	0.00	0.
1	10	12	0.00	0.00	0.
1	10	21	0.00	0.00	0.
1	10	30	0.00	0.00	0.
1	10	39	0.00	0.00	0.
1	10	48	0.00	0.00	0.
[illegible]	[illegible]	57	[illegible]	0.00	0.

1	11	24	0.00	0.00	0.
1	11	33	0.00	0.00	0.
1	11	42	0.00	0.00	0.
1	11	51	0.00	0.00	0.
1	11	60	0.00	0.00	0.
1	12	9	0.00	0.00	0.
1	12	18	0.00	0.00	0.
1	12	27	0.00	0.00	0.
1	12	36	0.00	0.00	0.
1	12	45	0.00	0.00	0.
1	12	54	0.00	0.00	0.
1	13	3	0.00	0.00	0.
1	13	12	0.00	0.00	0.
1	13	21	0.00	0.00	0.
1	13	30	0.00	0.00	0.
1	13	39	0.00	0.00	0.
1	13	48	0.00	0.00	0.
1	13	57	0.00	0.00	0.
1	14	6	0.00	0.00	0.
1	14	15	0.00	0.00	0.
1	14	24	0.00	0.00	0.
1	14	33	0.00	0.00	0.
1	14	42	0.00	0.00	0.
1	14	51	0.00	0.00	0.
1	14	60	0.00	0.00	0.
		SUM	18.16	18.16	147842.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	11688.	3662.	1478.	1479.	147841.
INCHES		18.12	18.29	18.29	18.29
AC-FT		1817.	1834.	1834.	1834.

OVV

********** ********** ********** ********** **********

HYDROGRAPH ROUTING

ROUTING COMPUTATION THROUGH RESERVIOR

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
2	1	0	0	0	0	1

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME
0.0	0.000	0.00	1	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA
0	0	0	0.000	0.000	0.000	-1.

STORAGE=	0.	39.	178.	269.	361.	454.	548.	644.	692.	740.
OUTFLOW=	0.	130.	424.	779.	1056.	1250.	1417.	7317.	12200.	17967.

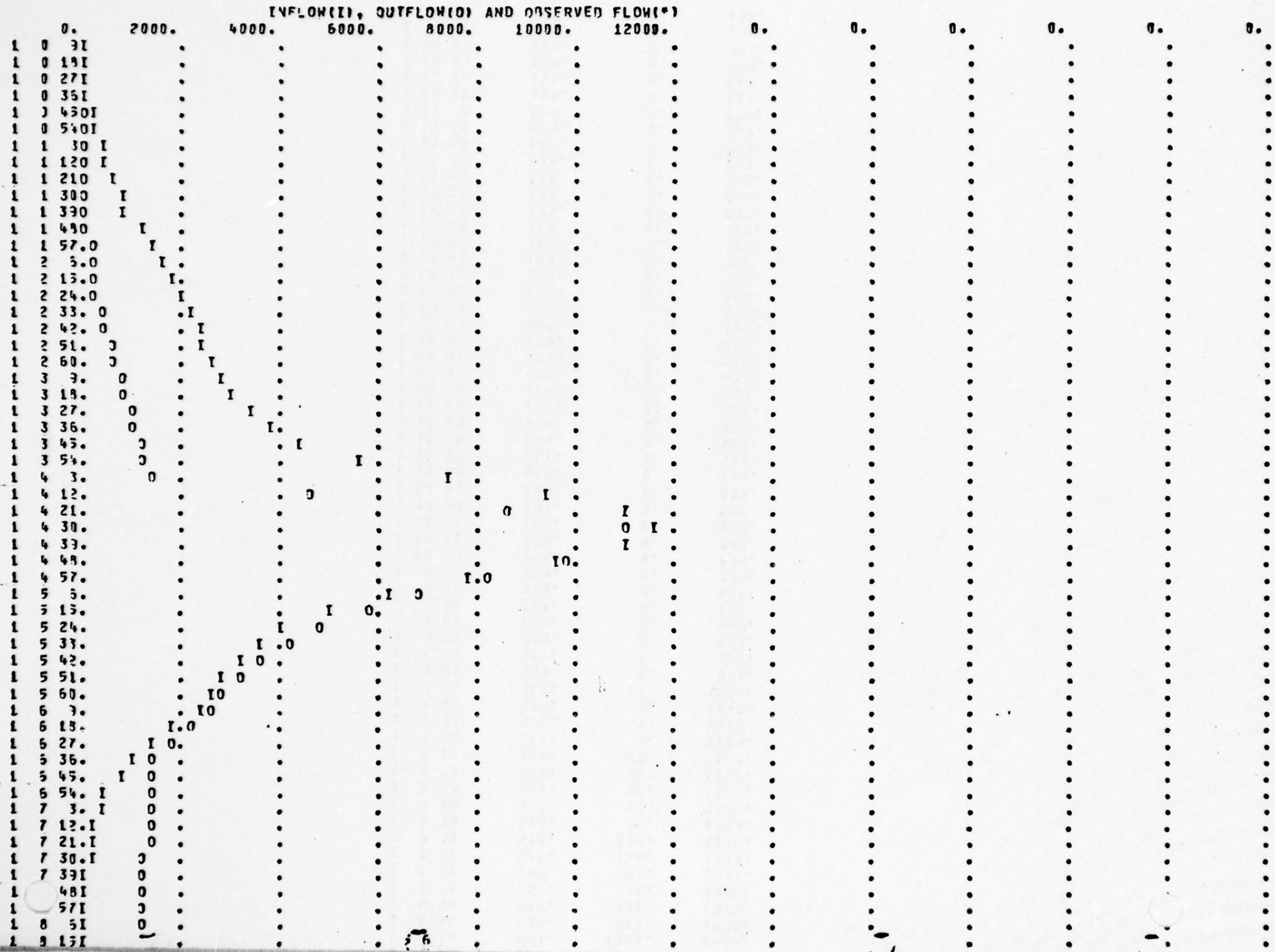
	TIME		EOP STOR	AVG IN	EOP OUT
1	0	3	0.	0.	0.
1	0	18	0.	4.	0.
1	0	27	0.	20.	1.
1	0	36	1.	59.	2.
1	0	45	3.	129.	4.
1	0	54	5.	227.	9.
1	1	3	9.	342.	16.
1	1	12	15.	462.	25.
1	1	21	22.	580.	37.
1	1	30	30.	716.	51.
1	1	39	40.	892.	68.
1	1	48	53.	1122.	90.
1	1	57	69.	1373.	116.
1	2	6	87.	1605.	147.
1	2	15	107.	1803.	207.
1	2	24	129.	1986.	273.
1	2	33	152.	2150.	344.
1	2	42	176.	2312.	417.
1	2	51	201.	2457.	512.
1	2	60	226.	2603.	611.
1	3	9	252.	2799.	714.
1	3	18	281.	3080.	815.
1	3	27	312.	3397.	910.
1	3	36	348.	3794.	1016.
1	3	45	389.	4440.	1115.
1	3	54	444.	5595.	1230.
1	4	3	519.	7356.	1366.
1	4	12	599.	9413.	4566.
1	4	21	656.	11084.	8501.
1	4	30	680.	11676.	10357.
1	4	39	680.	10995.	10986.
1	4	48	663.	9547.	9973.
1	4	57	653.	7734.	8219.
1	5	6	636.	6112.	6798.
1	5	15	619.	4959.	5783.
1	5	24	604.	4068.	4837.
1	5	33	592.	3509.	4104.
1	5	42	583.	3132.	3568.
1	5	51	577.	2861.	3178.
1	5	60	572.	2629.	2975.

1	6	36	547.	1016.	1414.
1	6	45	538.	710.	1399.
1	6	54	527.	471.	1379.
1	7	3	514.	320.	1356.
1	7	12	500.	217.	1331.
1	7	21	485.	146.	1305.
1	7	30	470.	100.	1279.
1	7	39	455.	70.	1253.
1	7	48	441.	47.	1222.
1	7	57	426.	32.	1192.
1	8	6	412.	22.	1162.
1	8	15	398.	15.	1133.
1	8	24	384.	9.	1104.
1	8	33	371.	5.	1076.
1	8	42	357.	2.	1045.
1	8	51	345.	1.	1007.
1	8	60	333.	0.	970.
1	9	9	321.	0.	935.
1	9	18	309.	0.	900.
1	9	27	298.	0.	867.
1	9	36	288.	0.	836.
1	9	45	278.	0.	805.
1	9	54	268.	0.	775.
1	10	3	258.	0.	738.
1	10	12	250.	0.	703.
1	10	21	241.	0.	670.
1	10	30	233.	0.	638.
1	10	39	225.	0.	608.
1	10	48	218.	0.	579.
1	10	57	211.	0.	552.
1	11	6	204.	0.	526.
1	11	15	198.	0.	501.
1	11	24	192.	0.	478.
1	11	33	186.	0.	455.
1	11	42	180.	0.	433.
1	11	51	175.	0.	415.
1	11	60	170.	0.	400.
1	12	9	165.	0.	385.
1	12	18	161.	0.	370.
1	12	27	156.	0.	356.
1	12	36	152.	0.	343.
1	12	45	148.	0.	330.
1	12	54	144.	0.	318.
1	13	3	140.	0.	306.
1	13	12	136.	0.	295.
1	13	21	132.	0.	284.
1	13	30	129.	0.	273.
1	13	39	126.	0.	263.
1	13	48	122.	0.	253.
1	13	57	119.	0.	243.
1	14	6	116.	0.	234.
1	14	15	114.	0.	225.
1	14	24	111.	0.	217.
1	14	33	108.	0.	209.
1	14	42	106.	0.	201.
1	14	51	103.	0.	194.
1	14	60	101.	0.	186.
	SUM				139803.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	10995.	2972.	1398.	1398.	139803.
INCHES		14.71	17.29	17.29	17.29
AC-FT		1474.	1734.	1734.	1734.

"OVE"

STATION 2



COMPUTER PRINTOUT

FOR ONE HALF OF THE

PROBABLE MAXIMUM FLOOD

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****************************
HEC-     ERSION DATED JAN 1973
UPDATED AUG 74
CHANGE NO. 01
****************************

                         .5
              ROUTING OF^PMF FLOOD
               PACKANACK LAKE DAM
       O-BRIEN + GERE--JUSTIN + COURINEY DIV

                  JOB SPECIFICATION
   NO   NHR  NMIN  IDAY   IHR  IMIN METRC  IPLT  IPRT NSTAN
  100     0     9     1     0     0     0     2     2     0
                    JOPER   NWT
                        3     0


**********   **********   **********   **********   **********

              SUB-AREA RUNOFF COMPUTATION

  PMF INFLOW HYDROGRAPH TO PACKANACK LAKE
       ISTAQ   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME
           1       0       0       0       0       0       1

                    HYDROGRAPH DATA
 IHYDG   IUHG  TAREA   SNAP  TRSDA  TRSPC  RATIO  ISNOW  ISAME  LOCAL
     6     -1   1.88   0.00   0.00   0.00  0.000      0      0      0

                    PRECIP DATA
              NP   STORM     DAJ     DAK
              37    0.00    0.00    0.00
                  PRECIP PATTERN
   .05    0.00     .15    0.00     .20    0.00     .26    0.00     .50    0.00
   .55    0.00     .60    0.00     .70    0.00     .70    0.00    1.05    0.00
  1.10    0.00    2.40    0.00    4.50    0.00    2.65    0.00     .55    0.00
   .55    0.00     .55    0.00     .55    0.00     .55

                     LOSS DATA
 STRKR  DLTKR  RTIOL  ERAIN  STRKS  RTIOK  STRTL  CNSTL  ALSMX  RTIMP
  0.00   0.00   1.00   0.00   0.00   1.00   0.00   0.00   0.00   0.00

             GIVEN UNIT GRAPH, NUMGQ=  23
    0.     85.    320.    612.    746.    650.    492.    385.    238.    164.
  116.     78.     56.     40.     27.     19.     13.     10.      7.      5.
    3.      1.      0.
   UNIT GRAPH TOTALS   4067. CFS OR   .50 INCHES OVER THE AREA

                  RECESSION DATA
          STRTQ=    0.00   QRCSN=    0.00   RTIOR= 1.00

               END-OF-PERIOD FLOW
         TIME    RAIN    EXCS    COMP Q
       1  0  9    .05     .05       0.
       1  0 18   0.00    0.00       4.
       1  0 27    .15     .15      16.
       1  0 36   0.00    0.00      43.
       1  0 45    .20     .20      85.
       1  0 54   0.00    0.00     141.
       1  1  3    .26     .26     201.
       1  1 12   0.00    0.00     241.
```

1	1	21	.50	.50	318.
1	1	30	0.00	0.00	308.
1	1	39	.55	.55	404.
1	1	48	0.00	0.00	627.
1	1	57	.60	.60	745.
1	2	6	0.00	0.00	859.
1	2	15	.70	.70	943.
1	2	24	0.00	0.00	1042.
1	2	33	.70	.70	1107.
1	2	42	0.00	0.00	1203.
1	2	51	1.05	1.05	1253.
1	2	60	0.00	0.00	1349.
1	3	9	1.10	1.10	1448.
1	3	18	0.00	0.00	1630.
1	3	27	2.40	2.40	1765.
1	3	36	0.00	0.00	2026.
1	3	45	4.50	4.50	2410.
1	3	54	0.00	0.00	3181.
1	4	3	2.65	2.65	4170.
1	4	12	0.00	0.00	5237.
1	4	21	.55	.55	5841.
1	4	30	0.00	0.00	5830.
1	4	39	.55	.55	5158.
1	4	48	0.00	0.00	4384.
1	4	57	.55	.55	3345.
1	5	6	0.00	0.00	2761.
1	5	15	.55	.55	2194.
1	5	24	0.00	0.00	1870.
1	5	33	.55	.55	1636.
1	5	42	0.00	0.00	1492.
1	5	51	0.00	0.00	1364.
1	5	60	0.00	0.00	1261.
1	6	9	0.00	0.00	1057.
1	6	18	0.00	0.00	833.
1	6	27	0.00	0.00	584.
1	6	36	0.00	0.00	429.
1	6	45	0.00	0.00	290.
1	6	54	0.00	0.00	189.
1	7	3	0.00	0.00	128.
1	7	12	0.00	0.00	86.
1	7	21	0.00	0.00	58.
1	7	30	0.00	0.00	41.
1	7	39	0.00	0.00	28.
1	7	48	0.00	0.00	19.
1	7	57	0.00	0.00	13.
1	8	6	0.00	0.00	9.
1	8	15	0.00	0.00	6.
1	8	24	0.00	0.00	3.
1	8	33	0.00	0.00	2.
1	8	42	0.00	0.00	1.
1	8	51	0.00	0.00	0.
1	8	60	0.00	0.00	0.
1	9	9	0.00	0.00	0.
1	9	18	0.00	0.00	0.
1	9	27	0.00	0.00	0.
1	9	36	0.00	0.00	0.
1	9	45	0.00	0.00	0.
1	9	54	0.00	0.00	0.
1	10	3	0.00	0.00	0.
1	10	12	0.00	0.00	0.
1	10	21	0.00	0.00	0.
1	10	30	0.00	0.00	0.

AD-A056 563 O'BRIEN AND GERE ENGINEERS INC PHILADELPHIA PA JUSTIN--ETC F/G 13/2
PHASE I INSPECTION REPORT. NATIONAL DAM SAFETY PROGRAM. PACKANA--ETC(U)
FEB 78 J J WILLIAMS DACW61-78-C-0052

UNCLASSIFIED NL

2 OF 2

AD
A056563

END
DATE
FILMED
9-78
DDC

1	10	39	0.00	0.00	0.	
1	10	48	0.00	0.00	0.	
1	10	57	0.00	0.00	0.	
1	11	6	0.00	0.00	0.	
1	11	15	0.00	0.00	0.	
1	11	24	0.00	0.00	0.	
1	11	33	0.00	0.00	0.	
1	11	42	0.00	0.00	0.	
1	11	51	0.00	0.00	0.	
1	11	60	0.00	0.00	0.	
1	12	9	0.00	0.00	0.	
1	12	18	0.00	0.00	0.	
1	12	27	0.00	0.00	0.	
1	12	36	0.00	0.00	0.	
1	12	45	0.00	0.00	0.	
1	12	54	0.00	0.00	0.	
1	13	3	0.00	0.00	0.	
1	13	12	0.00	0.00	0.	
1	13	21	0.00	0.00	0.	
1	13	30	0.00	0.00	0.	
1	13	39	0.00	0.00	0.	
1	13	48	0.00	0.00	0.	
1	13	57	0.00	0.00	0.	
1	14	6	0.00	0.00	0.	
1	14	15	0.00	0.00	0.	
1	14	24	0.00	0.00	0.	
1	14	33	0.00	0.00	0.	
1	14	42	0.00	0.00	0.	
1	14	51	0.00	0.00	0.	
1	14	60	0.00	0.00	0.	
		SUM	18.16	18.16	73856.	

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	5841.	1829.	739.	739.	73857.
INCHES		9.05	9.14	9.14	9.14
AC-FT		908.	916.	916.	916.

OVN

********** ********** ********** ********** **********

HYDROGRAPH ROUTING

ROUTING COMPUTATION THROUGH RESERVIOR

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
2	1	0	0	0	0	1

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME
0.0	0.000	0.00	1	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA
0	0	0	0.000	0.000	0.000	-1.

STORAGE=	0.	89.	178.	269.	361.	454.	548.	644.	692.	740.
OUTFLOW=	0.	150.	474.	779.	1056.	1250.	1417.	7317.	12200.	17967.

	TIME	EOP STOR	AVG IN	EOP OUT
1	0 9	0.	0.	0.
1	0 18	0.	2.	0.
1	0 27	0.	10.	0.
1	0 36	1.	30.	1.
1	0 45	1.	64.	2.
1	0 54	3.	113.	4.
1	1 3	5.	171.	8.
1	1 12	7.	231.	13.
1	1 21	11.	290.	18.
1	1 30	15.	358.	25.
1	1 39	20.	446.	34.
1	1 48	27.	561.	45.
1	1 57	34.	686.	58.
1	2 6	44.	802.	73.
1	2 15	54.	901.	91.
1	2 24	65.	993.	109.
1	2 33	77.	1075.	129.
1	2 42	89.	1155.	151.
1	2 51	102.	1228.	191.
1	2 60	116.	1301.	233.
1	3 9	130.	1398.	276.
1	3 18	145.	1539.	324.
1	3 27	162.	1698.	375.
1	3 36	181.	1896.	434.
1	3 45	202.	2218.	518.
1	3 54	230.	2796.	626.
1	4 3	267.	3675.	770.
1	4 12	315.	4703.	916.
1	4 21	371.	5539.	1077.
1	4 30	429.	5836.	1198.
1	4 39	482.	5494.	1290.
1	4 48	524.	4771.	1375.
1	4 57	553.	3864.	1723.
1	5 6	565.	3053.	2457.
1	5 15	565.	2478.	2468.
1	5 24	561.	2032.	2228.
1	5 33	557.	1753.	1966.
1	5 42	553.	1564.	1744.
1	5 51	550.	1428.	1570.
1	5 60	548.	1312.	[illegible]

A45

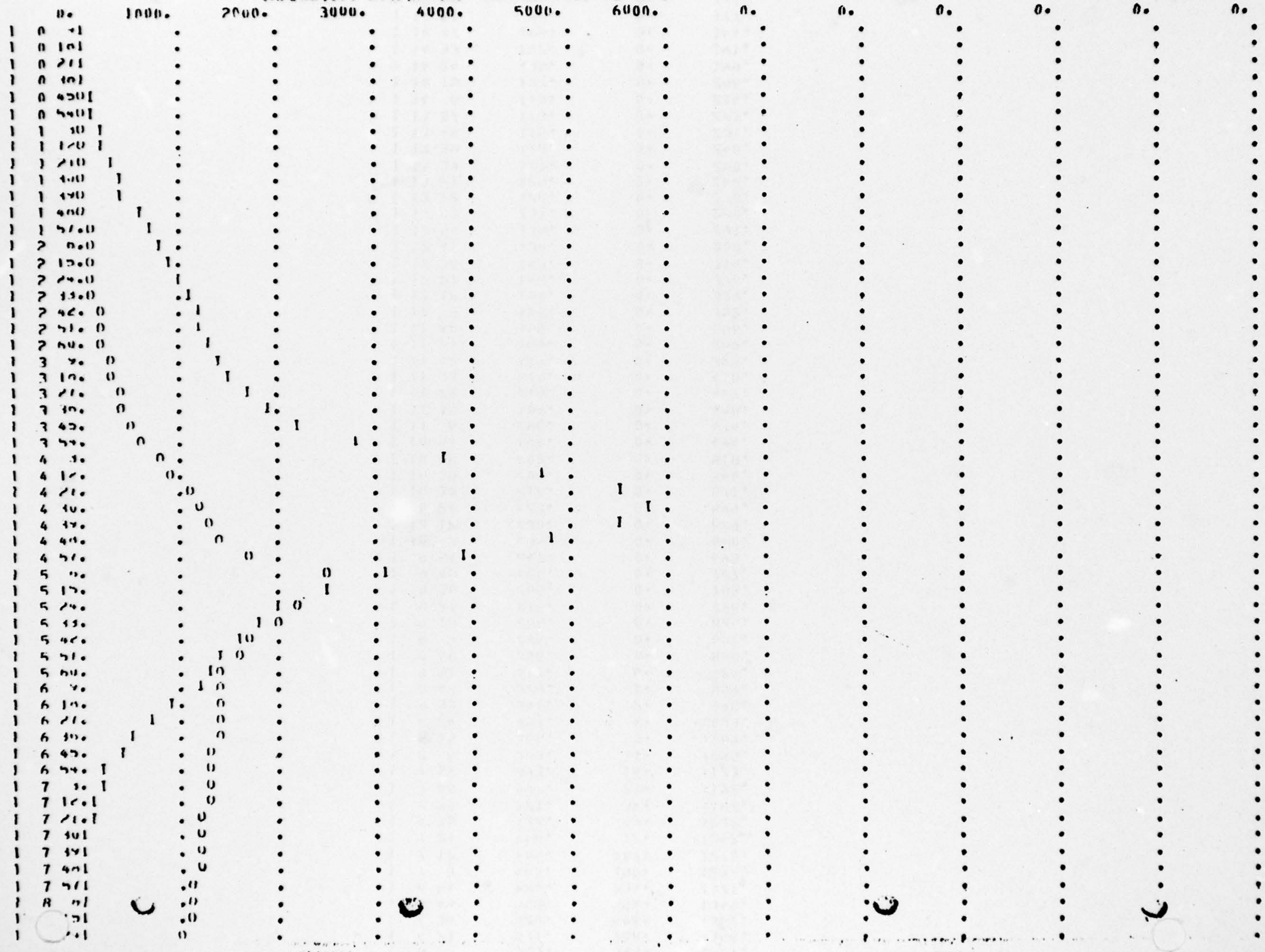
1	6	9	545.	1159.	1412.
1	6	18	539.	945.	1401.
1	6	27	531.	708.	1386.
1	6	36	520.	506.	1367.
1	6	45	508.	354.	1345.
1	6	54	494.	234.	1321.
1	7	3	480.	159.	1296.
1	7	12	465.	107.	1270.
1	7	21	450.	72.	1242.
1	7	30	436.	50.	1212.
1	7	39	421.	34.	1182.
1	7	48	407.	23.	1152.
1	7	57	393.	16.	1123.
1	8	6	380.	11.	1095.
1	8	15	366.	7.	1067.
1	8	24	353.	4.	1033.
1	8	33	341.	2.	995.
1	8	42	329.	1.	959.
1	8	51	317.	0.	924.
1	8	60	306.	0.	890.
1	9	9	295.	0.	857.
1	9	18	285.	0.	826.
1	9	27	275.	0.	796.
1	9	36	265.	0.	763.
1	9	45	256.	0.	727.
1	9	54	247.	0.	693.
1	10	3	238.	0.	660.
1	10	12	230.	0.	629.
1	10	21	223.	0.	599.
1	10	30	216.	0.	571.
1	10	39	209.	0.	544.
1	10	48	202.	0.	518.
1	10	57	196.	0.	494.
1	11	6	190.	0.	470.
1	11	15	184.	0.	448.
1	11	24	179.	0.	427.
1	11	33	174.	0.	410.
1	11	42	169.	0.	395.
1	11	51	164.	0.	380.
1	11	60	159.	0.	366.
1	12	9	155.	0.	352.
1	12	18	150.	0.	339.
1	12	27	146.	0.	326.
1	12	36	142.	0.	314.
1	12	45	138.	0.	302.
1	12	54	135.	0.	291.
1	13	3	131.	0.	280.
1	13	12	128.	0.	270.
1	13	21	125.	0.	260.
1	13	30	121.	0.	250.
1	13	39	118.	0.	240.
1	13	48	115.	0.	231.
1	13	57	113.	0.	223.
1	14	6	110.	0.	214.
1	14	15	107.	0.	206.
1	14	24	105.	0.	199.
1	14	33	102.	0.	191.
1	14	42	100.	0.	184.
1	14	51	98.	0.	177.
1	14	60	96.	0.	171.

SUM 762[illegible]

OVF

STATION 2

INFLOW(I), OUTFLOW(O) AND OBSERVED FLOW(*)



RECOMMENDED GUIDELINES

FOR SAFETY INSPECTION OF DAMS

CHAPTER 4

CHAPTER 4 - PHASE II INVESTIGATION

4.1. Purpose. The Phase II investigation will be supplementary to Phase I and should be conducted when the results of the Phase I investigation indicate the need for additional in-depth studies, investigations or analyses.

4.2. Scope. The Phase II investigation should include all additional studies, investigations and analyses necessary to evaluate the safety of the dam. Included, as required, will be additional visual inspections, measurements, foundation exploration and testing, materials testing, hydraulic and hydrologic analysis and structural stability analyses.

4.3. Hydraulic and Hydrologic Analysis. Hydraulic and hydrologic capabilities should be determined using the following criteria and procedures. Depending on the project characteristics, either the spillway design flood peak inflow or the spillway design flood hydrograph should be the basis for determining the maximum water surface elevation and maximum outflow. If the operation or failure of upstream water control projects would have significant impact on peak flow or hydrograph analyses, the impact should be assessed.

4.3.1. Maximum Water Surface Based on SDF Peak Inflow. When the total project discharge capability at maximum pool exceeds the peak inflow of the recommended SDF, and operational constraints would not prevent such a release at controlled projects, a reservoir routing is not required. The maximum discharge should be assumed equal to the peak inflow of the spillway design flood. Flood volume is not controlling in this situation and surcharge storage is either absent or is significant only to the extent that it provides the head necessary to develop the release capability required.

4.3.1.1. Peak for 100-Year Flood. When the 100-year flood is applicable under the provisions of Table 3 and data are available, the spillway design flood peak inflow may be determined by use of "A Uniform Technique for Determining Flood Frequencies," Water Resources Council (WRC), Hydrology Committee, Bulletin 15, December 1967. Flow frequency information from regional analysis is generally preferred over single station results when available and appropriate. Rainfall-runoff techniques may be necessary when there are inadequate runoff data available to make a reasonable estimate of flow frequency.

4.3.1.2. Peak for PMF or Fraction Thereof. When either the Probable Maximum Flood peak or a fraction thereof is applicable under the provisions of Table 3, the unit hydrograph - infiltration loss technique is generally the most expeditious method of computing the spillway design flood peak for most projects. This technique is discussed in the following paragraph.

4.3.2. Maximum Water Surface Based on SDF Hydrograph. Both peak and volume are required in this analysis. Where surcharge storage is significant, or where there is insufficient discharge capability at maximum pool to pass the peak inflow of the SDF, considering all possible operational constraints, a flood hydrograph is required. When there are upstream hazard areas that would be imperiled by fast rising reservoirs levels, SDF hydrographs should be routed to ascertain available time for warning and escape. Determination of probable maximum precipitation or 100-year precipitation, whichever is applicable, and unit hydrographs or runoff models will be required, followed by the determination of the PMF or 100-year flood. Conservative loss rates (significantly reduced by antecedent rainfall conditions where appropriate) should be estimated for computing the rainfall excess to be utilized with unit hydrographs. Rainfall values are usually arranged with gradually ascending and descending rates with the maximum rate late in the storm. When applicable, conservatively high snowmelt runoff rates and appropriate releases from upstream projects should be assumed. The PMP may be obtained from National Weather Service (NWS) publications such as Hydrometeorological Report (HMR) 33. Special NWS publications for particular areas should be used when available. Rainfall for the 100-year frequency flood can be obtained from the NWS publication "Rainfall Frequency Atlas of the United States," Technical Paper No. 40; Atlas 2, "Precipitation Frequency Atlas of Western United States;" or other NWS publications. The maximum water surface elevation and spillway design flood outflow are then determined by routing the inflow hydrograph through the reservoir surcharge storage, assuming a starting water surface at the bottom of surcharge storage, or lower when appropriate. For projects where the bottom of surcharge space is not distinct, or the flood control storage space (exclusive of surcharge) is appreciable, it may be appropriate to select starting water surface elevations below the top of the flood control storage for routings. Conservatively high starting levels should be estimated on the basis of hydrometeorological conditions reasonably characteristic for the region and flood release capability of the project. Necessary adjustment of reservoir storage capacity due to existing or future sediment or other encroachment may be approximated when accurate determination of deposition is not practicable.

4.3.3. Acceptable Procedures. Techniques for performing hydraulic and hydrologic analyses are generally available from publications prepared by Federal agencies involved in water resources development or textbooks written by the academic community. Some of these procedures are rather sophisticated and require expensive computational equipment and large data banks. While results of such procedures are generally more reliable than simplified methods, their use is generally not warranted in studies connected with this program unless they can be performed quickly and inexpensively. There may be situations where the more complex techniques have to be employed to obtain reliable results; however, these cases will be exceptions rather than the rule. Whenever the acceptability of procedures is in question, the advice of competent experts should be sought. Such expertise is generally available in the Corps of Engineers, Bureau of

Reclamation and Soil Conservation Service. Many other agencies, educational facilities and private consultants can also provide expert advice. Regardless of where such expertise is based, the qualification of those individuals offering to provide it should be carefully examined and evaluated.

4.3.4. Freeboard Allowances. Guidelines on specific minimum freeboard allowances are not considered appropriate because of the many factors involved in such determinations. The investigator will have to assess the critical parameters for each project and develop its minimum requirement. Many projects are reasonably safe without freeboard allowance because they are designed for overtopping, or other factors minimize possible overtopping. Conversely, freeboard allowances of several feet may be necessary to provide a safe condition. Parameters that should be considered include the duration of high water levels in the reservoir during the design flood; the effective wind fetch and reservoir depth available to support wave generation; the probability of high wind speed occurring from a critical direction; the potential wave runup on the dam based on roughness and slope; and the ability of the dam to resist erosion from overtopping waves.

4.4. Stability Investigations. The Phase II stability investigations should be compatible with the guidelines of this paragraph.

4.4.1. Foundation and Material Investigations. The scope of the foundation and materials investigation should be limited to obtaining the information required to analyze the structural stability and to investigate any suspected condition which would adversely affect the safety of the dam. Such investigations may include borings to obtain concrete, embankment, soil foundation, and bedrock samples; testing specimens from these samples to determine the strength and elastic parameters of the materials, including the soft seams, joints, fault gouge and expansive clays or other critical materials in the foundation; determining the character of the bedrock including joints, bedding planes, fractures, faults, voids and caverns, and other geological irregularities; and installing instruments for determining movements, strains, suspected excessive internal seepage pressures, seepage gradients and uplift forces. Special investigations may be necessary where suspect rock types such as limestone, gypsum, salt, basalt, claystone, shales or others are involved in foundations or abutments in order to determine the extent of cavities, piping or other deficiencies in the rock foundation. A concrete core drilling program should be undertaken only when the existence of significant structural cracks is suspected or the general qualitative condition of the concrete is in doubt. The tests of materials will be necessary only where such data are lacking or are outdated.

4.4.2. Stability Assessment. Stability assessments should utilize in situ properties of the structure and its foundation and pertinent geologic

information. Geologic information that should be considered includes groundwater and seepage conditions; lithology, stratigraphy, and geologic details disclosed by borings, "as-built" records, and geologic interpretation; maximum past overburden at site as deduced from geologic evidence; bedding, folding and faulting; joints and joint systems; weathering; slickensides, and field evidence relating to slides, faults, movements and earthquake activity. Foundations may present problems where they contain adversely oriented joints, slickensides or fissured material, faults, seams of soft materials, or weak layers. Such defects and excess pore water pressures may contribute to instability. Special tests may be necessary to determine physical properties of particular materials. The results of stability analyses afford a means of evaluating the structure's existing resistance to failure and also the effects of any proposed modifications. Results of stability analyses should be reviewed for compatibility with performance experience when possible.

4.4.2.1. <u>Seismic Stability</u>. The inertial forces for use in the conventional equivalent static force method of analysis should be obtained by multiplying the weight by the seismic coefficient and should be applied as a horizontal force at the center of gravity of the section or element. The seismic coefficients suggested for use with such analyses are listed in Figures 1 through 4. Seismic stability investigations for all high hazard category dams located in Seismic Zone 4 and high hazard dams of the hydraulic fill type in Zone 3 should include suitable dynamic procedures and analyses. Dynamic analyses for other dams and higher seismic coefficients are appropriate if in the judgment of the investigating engineer they are warranted because of proximity to active faults or other reasons. Seismic stability investigations should utilize "state-of-the-art" procedures involving seismological and geological studies to establish earthquake parameters for use in dynamic stability analyses and, where appropriate, the dynamic testing of materials. Stability analyses may be based upon either time-history or response spectra techniques. The results of dynamic analyses should be assessed on the basis of whether or not the dam would have sufficient residual integrity to retain the reservoir during and after the greatest or most adverse earthquake which might occur near the project location.

4.4.2.2. <u>Clay Shale Foundation</u>. Clay shale is a highly overconsolidated sedimentary rock comprised predominantly of clay minerals, with little or no cementation. Foundations of clay shales require special measures in stability investigations. Clay shales, particularly those containing montmorillonite, may be highly susceptible to expansion and consequent loss of strength upon unloading. The shear strength and the resistance to deformation of clay shales may be quite low and high pore water pressures may develop under increase in load. The presence of slickensides in clay shales is usually an indication of low shear strength. Prediction

of field behavior of clay shales should not be based solely on results of conventional laboratory tests since they may be misleading. The use of peak shear strengths for clay shales in stability analyses may be unconservative because of nonuniform stress distribution and possible progressive failures. Thus the available shear resistance may be less than if the peak shear strength were mobilized simultaneously along the entire failure surface. In such cases, either greater safety factors or residual shear strength should be used.

4.4.3. Embankment Dams.

4.4.3.1. Liquefaction. The phenomenon of liquefaction of loose, saturated sands and silts may occur when such materials are subjected to shear deformation or earthquake shocks. The possibility of liquefaction must presently be evaluated on the basis of empirical knowledge supplemented by special laboratory tests and engineering judgment. The possibility of liquefaction in sands diminishes as the relative density increases above approximately 70 percent. Hydraulic fill dams in Seismic Zones 3 and 4 should receive particular attention since such dams are susceptible to liquefaction under earthquake shocks.

4.4.3.2. Shear Failure. Shear failure is one in which a portion of an embankment or of an embankment and foundation moves by sliding or rotating relative to the remainder of the mass. It is conventionally represented as occurring along a surface and is so assumed in stability analyses, although shearing may occur in a zone of substantial thickness. The circular arc or the sliding wedge method of analyzing stability, as pertinent, should be used. The circular arc method is generally applicable to essentially homogeneous embankments and to soil foundations consisting of thick deposits of fine-grained soil containing no layers significantly weaker than other strata in the foundation. The wedge method is generally applicable to rockfill dams and to earth dams on foundations containing weak layers. Other methods of analysis such as those employing complex shear surfaces may be appropriate depending on the soil and rock in the dam and foundation. Such methods should be in reputable usage in the engineering profession.

4.4.3.3. Loading Conditions. The loading conditions for which the embankment structures should be investigated are (I) Sudden drawdown from spillway crest elevation or top of gates, (II) Partial pool, (III) Steady state seepage from spillway crest elevation or top of gate elevation, and (IV) Earthquake. Cases I and II apply to upstream slopes only; Case III applies to downstream slopes; and Case IV applies to both upstream and downstream slopes. A summary of suggested strengths and safety factors are shown in Table 4.

TABLE 4

FACTORS OF SAFETY ⨍

Case	Loading Condition	Factor of Safety	Shear ⨍⨍ Strength	Remarks
I	Sudden drawdown from spillway crest or top of gates to minimum drawdown elevation.	1.2*	Minimum composite of R and S shear strengths See Figure 5.	Within the drawdown zone submerged unit weights of materials are used for computing forces resisting sliding and saturated unit weights are used for computing forces contributing to sliding.
II	Partial pool with assumed horizontal steady seepage saturation.	1.5	$\frac{R+S}{2}$ for $R < S$ S for $R > S$	Composite intermediate envelope of R and S shear strengths. See Figure 6.
III	Steady seepage from spillway crest or top of gates with K_h/K_v = 9 assumed**	1.5	Same as Case II	
IV	Earthquake (Cases II and III with seismic loading)	1.0	***	See Figures 1 through 4 for Seismic Coefficients.

⨍ Not applicable to embankments on clay shale foundation. Experience has indicated special problems in determination of design shear strengths for clay shale foundations and acceptable safety factors should be compatible with the confidence level in shear strength assumptions.

⨍⨍ Other strength assumptions may be used if in common usage in the engineering profession.

* The safety factor should not be less than 1.5 when drawdown rate and pore water pressure developed from flow nets are used in stability analyses.

** K_h/K_v is the ratio of horizontal to vertical permeability. A minimum of 9 is suggested for use in compacted embankments and alluvial sediments.

*** Use shear strength for case analyzed without earthquake. It is not necessary to analyze sudden drawdown for earthquake loading. Shear strength tests are classified according to the controlled drainage conditions maintained during the test. R tests are those in which specimen drainage is allowed during consolidation (or swelling) under initial stress conditions, but specimen drainage is not allowed during application of shearing stresses. S tests allow full drainage during initial stress application and shearing is at a slow rate so that complete specimen drainage is permitted during the complete test.

4.4.3.4. Safety Factors. Safety factors for embankment dam stability studies should be based on the ratio of available shear strength to developed shear strength, S_D:

$$S_D = \frac{C}{F.S.} + \sigma \frac{\tan \phi}{F.S.} \qquad (1)$$

C = cohesion

ϕ = angle of internal friction

σ = normal stress

The factors of safety listed in Table 4 are recommended as minimum acceptable. Final accepted factors of safety should depend upon the degree of confidence the investigating engineer has in the engineering data available to him. The consequences of a failure with respect to human life and property damage are important considerations in establishing factors of safety for specific investigations.

4.4.3.5. Seepage Failure. A critical uncontrolled underseepage or through seepage condition that develops during a rising pool can quickly reduce a structure which was stable under previous conditions, to a total structural failure. The visually confirmed seepage conditions to be avoided are (1) the exit of the phreatic surface on the downstream slope of the dam and (2) development of hydrostatic heads sufficient to create in the area downstream of the dam sand boils that erode materials by the phenomenon known as "piping" and (3) localized concentrations of seepage along conduits or through pervious zones. The dams most susceptible to seepage problems are those built of or on pervious materials of uniform fine particle size, with no provisions for an internal drainage zone and/or no underseepage controls.

4.4.3.6. Seepage Analyses. Review and modifications to original seepage design analyses should consider conditions observed in the field inspection and piezometer instrumentation. A seepage analysis should consider the permeability ratios resulting from natural deposition and from compaction placement of materials with appropriate variation between horizontal and vertical permeability. An underseepage analysis of the embankment should provide a critical gradient factor of safety for the maximum head condition of not less than 1.5 in the area downstream of the embankment.

$$F.S = i_c/i = \frac{H_c/D_b}{H/D_b} = D_b \frac{(\gamma_m - \gamma_w)}{H \gamma_w} \qquad (2)$$

i_c = Critical gradient

i = Design gradient

H = Uplift head at downstream toe of dam measured above tailwater

H_c = The critical uplift

D_b = The thickness of the top impervious blanket at the downstream toe of the dam

γ_m = The estimated saturated unit weight of the material in the top impervious blanket

γ_w = The unit weight of water

Where a factor of safety less than 1.5 is obtained the provision of an underseepage control system is indicated. The factor of safety of 1.5 is a recommended minimum and may be adjusted by the responsible engineer based on the competence of the engineering data.

4.4.4. Concrete Dams and Appurtenant Structures.

4.4.4.1. Requirements for Stability. Concrete dams and structures appurtenant to embankment dams should be capable of resisting overturning, sliding and overstressing with adequate factors of safety for normal and maximum loading conditions.

4.4.4.2. Loads. Loadings to be considered in stability analyses include the water load on the upstream face of the dam; the weight of the structure, internal hydrostatic pressures (uplift) within the body of the dam, at the base of the dam and within the foundation; earth and silt loads; ice pressure, seismic and thermal loads, and other loads as applicable. Where tailwater or backwater exists on the downstream side of the structure it should be considered, and assumed uplift pressures should be compatible with drainage provisions and uplift measurements if available. Where applicable, ice pressure should be applied to the contact surface of the structure at normal pool elevation. A unit pressure of not more than 5,000 pounds per square foot should be used. Normally, ice thickness should not be assumed greater than two feet. Earthquake forces should consist of the inertial forces due to the horizontal acceleration of the dam itself and hydrodynamic forces resulting from the reaction of the reservoir water against the structure. Dynamic water pressures for use in conventional methods of analysis may be computed by means of the "Westergaard Formula" using the parabolic approximation (H.M. Westergaard, "Water Pressures on Dams During Earthquakes," Trans., ASCE, Vol 98, 1933, pages 418-433), or similar method.

4.4.4.3. Stresses. The analysis of concrete stresses should be based on in situ properties of the concrete and foundation. Computed maximum compressive stresses for normal operating conditions in the order of 1/3 or less of in situ strengths should be satisfactory. Tensile stresses in unreinforced concrete should be acceptable only in locations where cracks will not adversely affect the overall performance and stability of the structure. Foundation stresses should be such as to provide adequate safety against failure of the foundation material under all loading conditions.

4.4.4.4. Overturning. A gravity structure should be capable of resisting all overturning forces. It can be considered safe against overturning if the resultant of all combinations of horizontal and vertical forces, excluding earthquake forces, acting above any horizontal plane through the structure or at its base is located within the middle third of the section. When earthquake is included the resultant should fall within the limits of the plane or base, and foundation pressures must be acceptable. When these requirements for location of the resultant are not satisfied the investigating engineer should assess the importance to stability of the deviations.

4.4.4.5. Sliding. Sliding of concrete gravity structures and of abutment and foundation rock masses for all types of concrete dams should be evaluated. by the shear-friction resistance concept. The available sliding resistance is compared with the driving force which tends to induce sliding to arrive at a sliding stability safety factor. The investigation should be made along all potential sliding paths. The critical path is that plane or combination of planes which offers the least resistance.

4.4.4.5.1. Sliding Resistance. Sliding resistance is a function of the unit shearing strength at no normal load (cohesion) and the angle of friction on a potential failure surface. It is determined by computing the maximum horizontal driving force which could be resisted along the sliding path under investigation. The following general formula is obtained from the principles of statics and may be derived by resolving forces parallel and perpendicular to the sliding plane:

$$R_R = V \tan(\phi + \alpha) + \frac{cA}{\cos\alpha(1 - \tan\phi \tan\alpha)} \qquad (3)$$

where

R_R = Sliding Resistance (maximum horizontal driving force which can be resisted by the critical path)

ϕ = Angle of internal friction of foundation material or, where applicable, angle of sliding friction

V = Summation of vertical forces (including uplift)

c = Unit shearing strength at zero normal loading along potential failure plane

A = Area of potential failure plane developing unit shear strength "c"

α = Angle between inclined plane and horizontal (positive for uphill sliding)

For sliding downhill the angle α is negative and Equation (1) becomes:

$$R_R = V \tan(\phi - \alpha) + \frac{cA}{\cos\alpha\ (1 + \tan\phi \tan\alpha)} \qquad (4)$$

When the plane of investigation is horizontal, and the angle α is zero and Equation (1) reduced to the following:

$$R_R = V \tan\phi + cA \qquad (5)$$

4.4.4.5.2. Downstream Resistance. When the base of a concrete structure is embedded in rock or the potential failure plane lies below the base, the passive resistance of the downstream layer of rock may sometimes be utilized for sliding resistance. Rock that may be subjected to high velocity water scouring should not be used. The magnitude of the downstream resistance is the lesser of (a) the shearing resistance along the continuation of the potential sliding plane until it daylights or (b) the resistance available from the downstream rock wedge along an inclined plane. The theoretical resistance offered by the passive wedge can be computed by a formula equivalent to formula (3):

$$P_p = W \tan(\phi + \alpha) + \frac{cA}{\cos\alpha\,(1 - \tan\phi \tan\alpha)} \qquad (6)$$

P_p = passive resistance of rock wedge

W = weight (buoyant weight if applicable) of downstream rock wedge above inclined plane of resistance, plus any superimposed loads

ϕ = angle of internal friction or, if applicable, angle of sliding friction

α = angle between inclined failure plane and horizontal

c = unit shearing strength at zero normal load along failure plane

A = area of inclined plane of resistance

When considering cross-bed shear through a relatively shallow, competent rock strut, without adverse jointing or faulting, W and α may be taken at zero and 45°, respectively, and an estimate of passive wedge resistance per unit width obtained by the following equation:

$$P_p = 2\,cD \qquad (7)$$

where

D = Thickness of the rock strut

4.4.4.5.3. Safety Factor. The shear-friction safety factor is obtained by dividing the resistance R_R by H, the summation of horizontal service

loads to be applied to the structure:

$$S_{s\text{-}f} = \frac{R_R}{H} \qquad (8)$$

When the downstream passive wedge contributes to the sliding resistance, the shear fruction safety factor formula becomes:

$$S_{s\text{-}f} = \frac{R_R + P_p}{H} \qquad (9)$$

The above direct superimposition of passive wedge resistance is valid only if shearing rigidities of the foundation components are similar. Also, the compressive strength and buckling resistance of the downstream rock layer must be sufficient to develop the wedge resistance. For example, a foundation with closely spaced, near horizontal, relatively weak seams might not contain sufficient buckling strength to develop the magnitude of wedge resistance computed from the cross-bed shear strength. In this case wedge resistance should not be assumed without resorting to special treatment (such as installing foundation anchors). Computed sliding safety factors approximating 3 or more for all loading conditions without earthquake, and 1.5 including earthquake, should indicate satisfactory stability, depending upon the reliability of the strength parameters used in the analyses. In some cases when the results of comprehensive foundation studies are available, smaller safety factors may be acceptable. The selection of shear strength parameters should be fully substantiated. The bases for any assumptions; the results of applicable testing, studies and investigations; and all pre-existing, pertinent data should be reported and evaluated.